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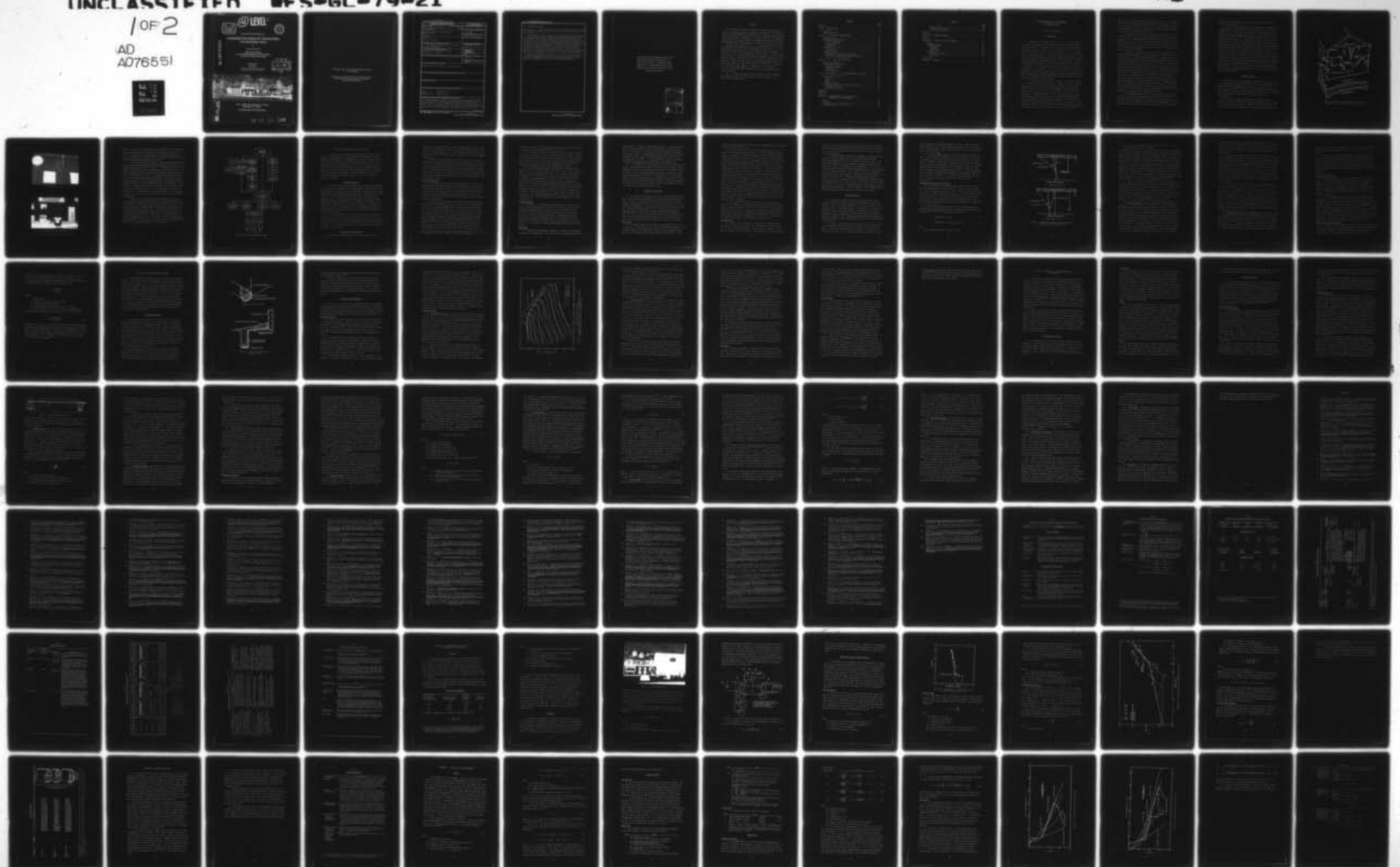

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OVERVIEW FOR DESIGN OF FOUNDATIONS ON EXPANSIVE SOILS

by

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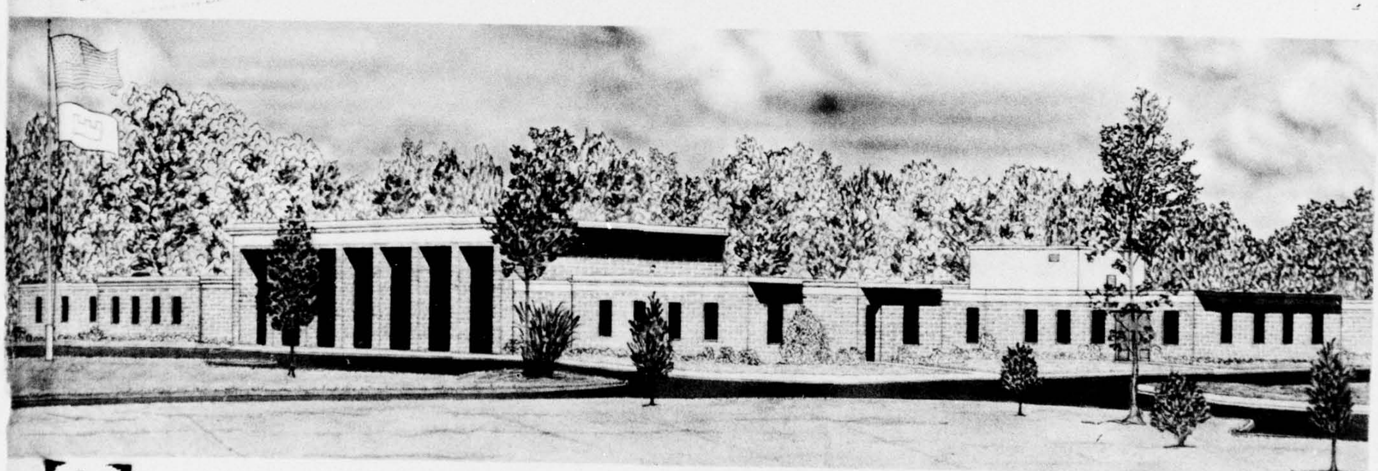

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Numerous structures constructed on expansive clay soil have experienced and sustained significant damage from differential heave and settlement. The types of structures most often damaged from heaving soil include highways, foundations and walls of residential and light commercial buildings, canal and reservoir linings, and retaining walls. The leading cause of foundation heave or settlement is change in soil moisture attributed to change in the field

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20. ABSTRACT (Continued).

environment (e.g., climatic changes, prevention of evaporation beneath covered areas, improper drainage following construction and from usage requirements of the structure).

The design process for structures on expansive clay should consist of a feasibility study, preliminary design phase to establish the overall concept, and a detailed design phase to complete the engineering description of the project. This report provides background information for establishing the preliminary design of structures in swelling soil areas based on field studies conducted by the U. S. Army Engineer Waterways Experiment Station (WES) and experiences of numerous investigators. The overview includes analyses of site and soil investigations, topography and landscaping including drainage and soil stabilization techniques, and selection of the foundation and superstructure. General suggestions for remedial repair of existing structures are also provided. Analyses of the movement of cast-in-place concrete piers in swelling soil are included to provide a basis for design of these foundations.

Appendices include:

Appendix A presents a determination of soil suction by thermocouple psychrometers. Suggestions for repair of structures (remedial measures) are presented in Appendix B. Prediction of pier movement is discussed in Appendix C. Appendix D is a notation of symbols used in the report.

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PREFACE

This overview for design of foundations on expansive soils is one phase in a continuing study of Research, Development, Test and Evaluation for Work Unit AT40 E0 004 "Foundations on Swelling Soils" sponsored by the Office, Chief of Engineers, U. S. Army. The report "Predicting Potential Heave and Heave with Time in Swelling Foundation Soils," Technical Report S-78-7, was completed July 1978 as part of this work unit.

The report was prepared by Dr. L. D. Johnson, Research Group (RG), Soil Mechanics Division (SMD), Geotechnical Laboratory (GL), U. S. Army Engineer Waterways Experiment Station (WES), CE, under the general supervision of Mr. C. L. McAnear, Chief, SMD, and Mr. J. P. Sale, Chief, GL. Messrs. W. R. Stroman, Foundations and Materials Branch, U. S. Army Engineer District, Fort Worth; F. H. Chen, President, Chen & Associates, Denver; Dr. John E. Holland, Principal Lecturer, Swinburne College of Technology, Melbourne, Australia; Messrs. G. B. Mitchell, Chief, Engineering Studies Branch, SMD; W. C. Sherman, Dr. E. B. Perry, and Dr. D. R. Snethen, RG, SMD, reviewed the report and contributed many helpful comments.

COL J. L. Cannon, CE, and COL N. P. Conover, CE, were Commanders and Directors of WES during the preparation of this report. Mr. F. R. Brown was Technical Director.

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CONTENTS

OVERVIEW FOR DESIGN OF FOUNDATIONS ON EXPANSIVE SOILS

PART I: INTRODUCTION

Background

1. Expansive clay foundation soils are located in many parts of the world, including much of the western, central, and southern areas of the United States.[1,2] Expansive soils, which swell or shrink substantially due to changes in water content, are characteristically highly plastic clays and clay shales that often contain colloidal clay minerals such as the montmorillonites. Numerous structures constructed on these soils, including many military facilities, have experienced and sustained significant damage from differential heave and settlement.[3-5] Differential movements redistribute loads of the structure on the elements of the foundation and can cause large changes in moments and shears not accounted for in the design.[6] These changes may also further aggravate differential movement and worsen damages to the structure. The types of structures most often damaged from heaving soil include foundations and walls of residential and light commercial buildings, highways, canal and reservoir linings, and retaining walls.

2. The leading cause of foundation heave or settlement is change in soil moisture, which is attributed to changes in the field environment from time of construction and usage requirements of the structure.[1,7,8] Other causes of soil volume changes are frost heave[9] and chemical reactions in the soil (e.g., oxidation of pyrite).[10,11,12] Structures on expansive foundation soils often heave because covered areas reduce the natural evaporation of moisture from the ground and reduce transpiration of moisture from vegetation. Construction on a site where a large tree was removed, for example, may lead to a buildup of moisture because of prior depletion of soil moisture by the extensive root system of the tree.[13] Additional changes in soil moisture are

attributed to significant variations in climate, such as long droughts and heavy rains, watering of lawns, depth to the water table, and inadequate drainage of surface water from the structure. Moisture changes also may be introduced into foundation soils through excavations made for basements or drilled pier foundations.

3. Differential heave can be caused by nonuniform changes in soil moisture and variations in thickness and composition of the expansive foundation soil. Nonuniform moisture changes occur from local concentrations of water from surface ponding, broken water and sewer lines, leaky faucets, defective rain gutters and downspouts, local transpiration of moisture from nearby trees, and diffusion of moisture away from heat sources such as furnaces.

4. Heaving of foundations is often erratic and associated with upward, long-term movements of four or more years. Movement that occurs from a reduction of natural evapotranspiration is commonly associated with a dome-shaped pattern of greatest movement toward the center of the structure, as documented in South Africa.[14-19] Localized heaving can be introduced at points where water leaks occur. In a structure undergoing generalized, widespread movement, a cyclic expansion-contraction related to drainage and the frequency and amount of rainfall and evapotranspiration is superimposed on long-term heave near the perimeter of the structure. Damaging end lift of foundations has been observed relatively soon after construction, which was associated with preconstruction vegetation and less topographic relief.[20] Downwarping from soil shrinkage may occur beneath the perimeter during hot, dry periods or from the desiccating effect of trees and vegetation adjacent to the structure.[20,21] Edge effects extend inward as much as 8 ft (2.5 m) and become less significant on well-drained land.[22-25]

5. A dish-shaped pattern can also occur beneath foundations due to consolidation, drying out of surface soil from a heat source, or lowering of the water table.[26,27] Damages are generally less in settling soil with the dish-shaped pattern because the foundation is usually better able to resist tension forces than the walls.[28] The semiarid,

hot and dry climates tend to cause the most severe and progressive foundation soil heaves.[29]

6. Types of damage sustained by structures due to differential vertical heave of foundation soil include distortion and cracking of pavements and on-grade floor slabs; cracks in grade beams, walls, and pier shafts; jammed or misaligned doors and windows; and failure of concrete plinths.[6,7,25,30,31] Lateral forces may lead to buckling of basement and retaining walls, particularly in overconsolidated and non-fissured soils. Figure 1 schematically illustrates some commonly observed exterior wall cracks from doming or edgedown patterns of heave. Typical fractures caused by movement of swelling soil beneath an abandoned structure near Clinton, Miss., are illustrated in Figure 2. The pattern of heave generally causes the external walls in the superstructure to lean outward, resulting in horizontal, vertical, and diagonal fractures with larger cracks near the top. The roof tends to restrain the rotation from vertical differential movements leading to additional horizontal fractures near the roofline at the top of the wall.[16,30-33] These damages can lead to difficult and costly long-term maintenance problems; *e.g.*, the maintenance expense of a single, military structure has exceeded $250,000.[5]

Purpose and Scope

7. Damages in structures founded on expansive soils occur because uniform and reliable design procedures are not generally available. Unsuitable design approaches that do not consider the potential of soil swell are often used.[24,34] Designs of relatively small structures such as residences and lightly loaded buildings, for example, are often based on local experience without adequate investigation of soil characteristics.

8. The design process sometimes omits but should consist of a feasibility study to establish the need and provide economic justification, preliminary design phase to establish the overall concept, and a detailed design phase to complete the engineering description of the

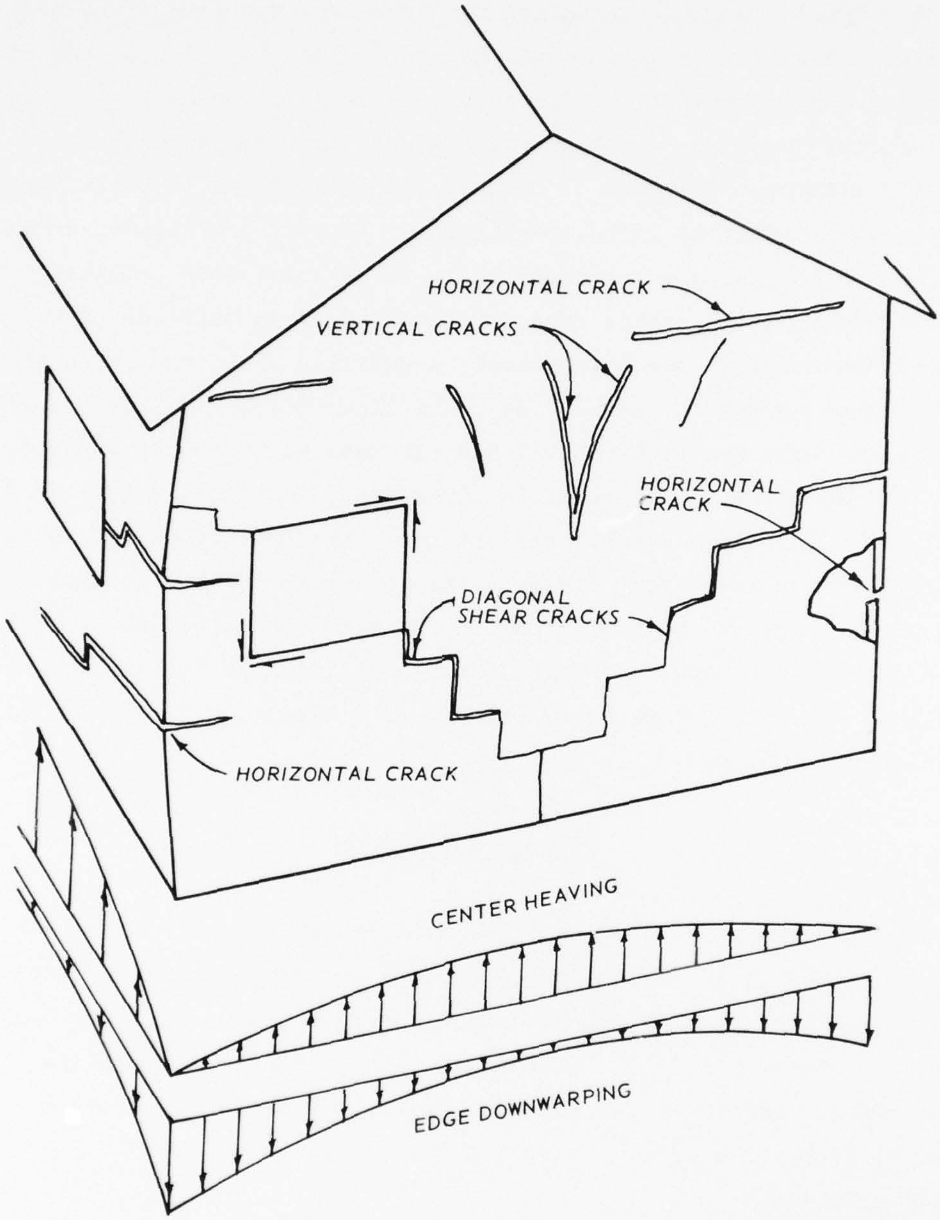
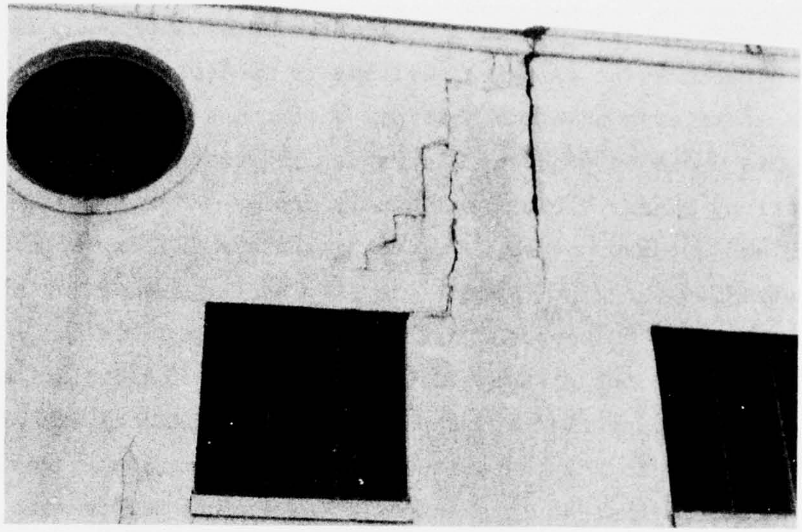


Figure 1. Examples of wall fractures from swelling foundation soils (after References 6, 30, 31)



a. Vertical cracks

b. Diagonal and vertical cracks

Figure 2. Examples of cracks in an exterior wall

project.[5] This report provides background information for establishing the preliminary design of structures in swelling soil areas with the intent to impart a basic understanding of successful procedures for design of structures on swelling soil and to present methods for anticipating and minimizing problems that may occur.

9. The decision process, Figure 3, illustrates interrelationships between various phases during preliminary design to properly select the foundation and superstructure. Figure 3 is a simplified version of the pattern methodology design concept proposed by Prendergast et al.[5] The pattern methodology concept shows that the design process includes site and soil investigatons, a study of topography and landscaping, and the selection of the foundation and superstructure. The decision concept is proposed partly to help determine during the preliminary design phase potential problems that could eventually affect the performance of the structure. Compromises can then be made between the structural, architectural, and mechanical aspects of the design without disrupting the design process. Changes during the detailed design phase or during construction are much more likely to delay construction and pose economic disadvantages.

10. The scope of this report includes analyses of site and soil investigations, topography and landscaping including drainage and soil stabilization techniques, and selection of the superstructure and foundation. Methods for remedial repairs of existing structures are also provided for reference (Appendix B). An analysis of the movement of cast-in-place pier foundations (Appendix C) is included as part of the procedure for selection of pier foundations to supplement the rather sketchy information available on the behavior of piers in swelling soil. The report does not specifically include procedures for design of highways, canal or reservoir linings, or retaining walls.

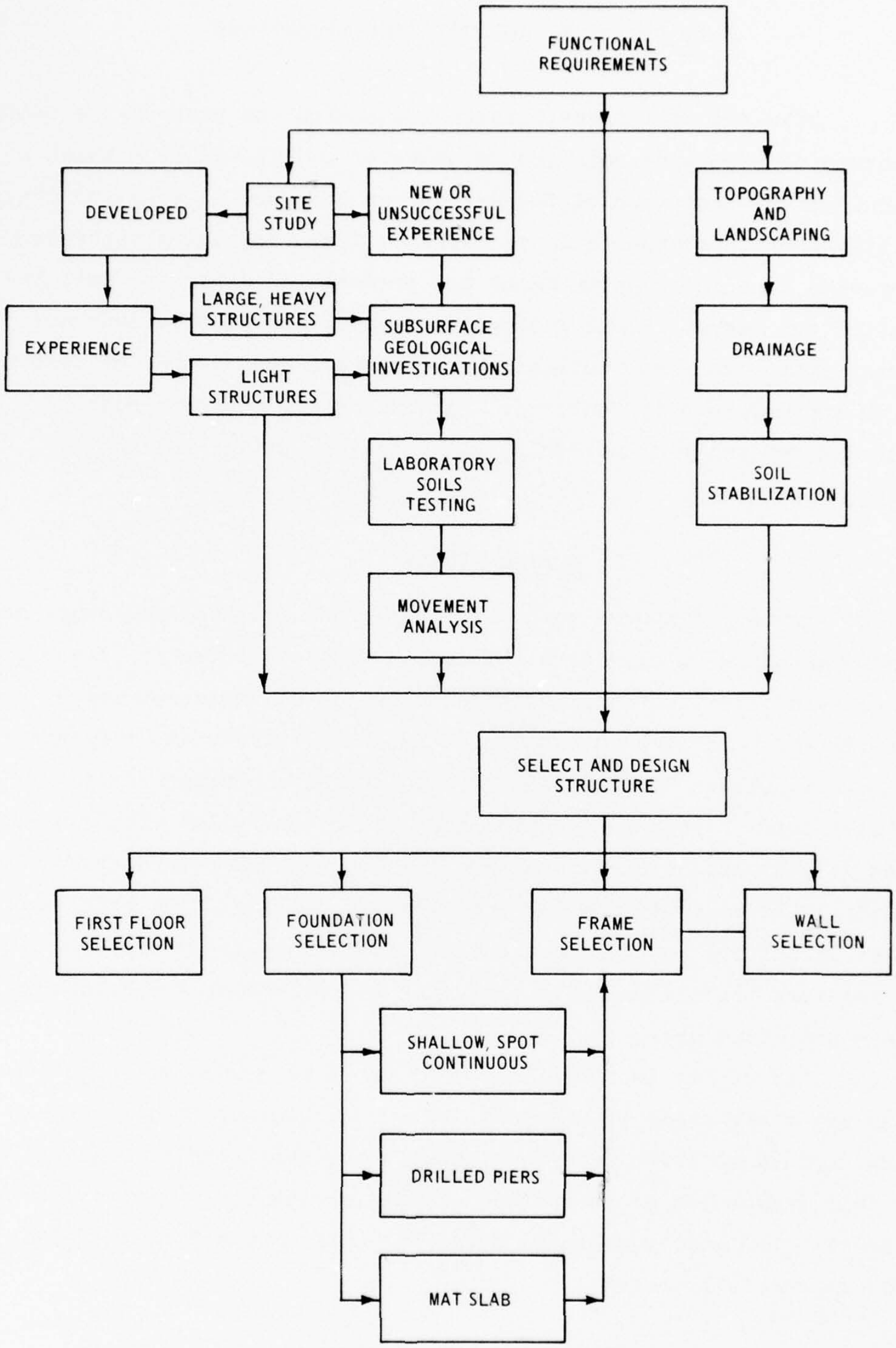


Figure 3. Decision process of design

PART II: SITE AND SOIL INVESTIGATIONS

11. Site and soil investigations determine the presence, extent, and nature of expansive soil and groundwater conditions from which a judgment of the best type of foundation can be made. A study of available literature, previous climate, surface features, and site history can provide much information about the presence of expansive soil and potential for heave. Local geological records and publications and federal, state, and institutional surveys are a good source of information on subsurface soil features. Meteorological records indicate amount and frequency of rainfall, which are useful for estimating climatic conditions.

Surface Features

12. Surface features such as wooded areas, bushes, and other deep-rooted vegetation in expansive soil areas indicate potential heave from accumulation of moisture following elimination of these sources of evapotranspiration. The growth of mesquite and small trees may indicate subsurface soil with a high affinity for moisture, a characteristic of expansive soil.[5] Ponds and depressions are often filled with clayey, expansive sediments accumulated from the drainage of rainwater.[34] The site should be examined for the presence of gilgaies. The existence of earlier structures on the construction site has probably modified the soil moisture profile and will influence the potential for future heave beneath new structures.

13. Structures in the vicinity of the site should be inspected for cracks and other signs of distress. The condition of on-site stucco facing, joints of brick and stone structures, and interior plaster walls is a fair indication of subsurface expansive clay and relative potential for heave. The most successful types of local foundations and designs should be carefully noted.

Subsurface Investigations

14. Subsurface investigations are especially important in

expansive soil areas because the effects of swelling soil on the structure should be evaluated as well as the effects of the structure on the behavior of the foundation soil.[5] The subsurface exploration program should determine the extent and nature of expansive soil and groundwater conditions.

15. The design of residences and light structures can often be made with minimal additional subsurface investigations and soil testing if the site is developed, subsurface features are generally known, and local practice has provided consistently successful designs for structures. Unsuccessful local practice should be investigated to determine the reasons for failure. New sites and the design of large, heavy buildings require subsurface investigations and soil testing programs as part of the design process.

Field explorations

16. Field explorations should include investigation of soils between ground surface and bottom of the footing as well as materials beneath the proposed depth of footing. The swelling of expansive soil, for example, causes lateral thrust on foundation walls and uplift forces on pier shafts and differential movement between the foundation and underground utilities such as water and sewer lines, storm drains, and electrical connections.[5]

17. Sampling to depths greater than for normal investigations is often useful in expansive soil areas. The depth of sampling should be at least as deep as the probable depth to which moisture changes will occur; i.e., the depth of the active zone X_a * for heave. The depth X_a is often difficult to predict without field measurements of heave or moisture changes, and X_a has also not been established for many practical cases. The active zone usually extends down about 10-13 ft (3-4 m) in depth or to the depth of a shallow water table, but can go deeper.[18,19,35-38] The entire thickness of intensely jointed clay shales should be drilled and sampled until the groundwater level is encountered because the entire zone could swell when given access to

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* Symbols are listed and defined in the Notation, Appendix D.

moisture.[5] The depth of such desiccated and stiff, fissured clay shales at Lackland Air Force Base exceeds 50 ft (15 m).[19,39]

18. A competent inspector or engineer should accurately and visually classify materials as they are recovered from the boring. Adequate classification ensures proper selection of samples for laboratory tests. A qualified engineering geologist or foundation engineer should closely monitor the drill crew so that timely adjustments can be made during drilling to obtain the best and most representative samples.

19. Undisturbed samples should be obtained at intervals of not greater than 5 ft (1.5 m) of depth. The outer 0.4 in. (1 cm) of material should be removed from the perimeter of the core sample if the sample was exposed to drilling fluid. A coating of wax should be brushed on the sample before wrapping with foil, plastic wrap, cheesecloth, etc. The initial brushed coating of wax reduces subsequent penetration of molten wax into fissures during the sample sealing procedure. The temperature of the molten wax, a 1-to-1 mixture of paraffin and microcrystalline wax, should be as low as possible to avoid driving moisture from the sample. The outer perimeter of the sample should be trimmed during preparation of specimens for laboratory tests, leaving the more undisturbed inner core. Further details on undisturbed sampling may be found in Reference 40.

Time of sampling

20. Moisture in soil samples should be similar to moisture conditions of the foundation soil at the time of construction to best simulate the swelling behavior of expansive soil from laboratory tests. Undisturbed samples preferably should be taken when soil moisture is expected to be similar during construction, or samples may be taken during the dry season when potential heave will be maximum, thus providing a more conservative design. Heave of foundation soil tends to be less if the structure is constructed immediately following the rainy season.

Groundwater

21. Knowledge of groundwater conditions is important in evaluating the behavior of a foundation. The active zone for moisture change often

extends down to the depth of shallow water tables. A shallow perched water table may provide a source of moisture into deeper desiccated zones if open boreholes or foundation elements penetrate through the perched water table. Footings bottomed below a perched table may heave if measures are not taken to inhibit the migration of moisture into soils beneath the footings. A rising water table may also contribute to heave if footings are bottomed above the groundwater level.

22. The distribution of pore pressures in normal and perched water tables is determined by piezometric installations at different depths. Casagrande (ceramic porous tube) piezometers with small diameter (3/8 in. or 10 mm) risers are usually adequate, and they are relatively simple, inexpensive, and good for soils of low permeability.[41] All boreholes should be filled and sealed with a low permeable grout, such as 12 percent bentonite and 88 percent cement by weight, to minimize penetration of surface water or water from perched tables down into deeper strata that may include desiccated expansive clays.

Laboratory Soil Tests

23. The purpose of laboratory tests is to determine physical properties that provide input parameters for evaluating foundation performance. Results of classification tests permit a rating of relative expansive characteristics, but the actual field environment is often not reflected and estimates of field heaves from these tests may be misleading. Commonly used classification tests include specific gravity, Atterberg limits, natural water content, gradation, and hydrometer tests.[42] Predictions of total and differential movement from results of swell tests have provided more acceptable data to help determine the best type of foundation and depth of footing to support the structure.

Swell tests

24. Recommended swell tests include consolidometer swell and soil suction tests. Consolidometer swell tests tend to predict minimal levels of heave, whereas soil suction tests tend to predict maximum or upper levels of heave compared with those measured in the field.[19,43] Soil

suction tests have been more economical, less time-consuming, and simpler than consolidometer swell tests.

25. The procedure often used for consolidometer swell tests is described in Technical Manual TM 5-818-1, 15 Aug 61, "Engineering and Design - Procedures for Foundation Design of Buildings and Other Structures (Except Hydraulic Structures)."[44] An appropriate test when little is known about swell behavior or groundwater conditions is the consolidometer test described in Engineer Manual EM 1110-2-1906,[42] except that distilled water should be added at the seating or lowest possible load rather than at 0.25 tsf (24 kPa). The specimen is allowed to expand at the seating load until primary swell is complete before applying the consolidation pressures. A loading pressure simulating field initial conditions should be applied at the start of the test to determine the initial void ratio, then removed to the seating load prior to adding the water. This procedure, similar to that proposed by Jennings et al.,[45] can help to avoid the need for additional unscheduled tests when swelling behavior is different than anticipated (e.g., the specimen consolidates rather than swells following addition of water at significant loading pressures).[46] The void ratio log pressure curve for final effective pressures from the seating to maximum applied load can be used to determine settlement or heave with respect to the initial void ratio. The rebound curve is not needed.

26. Soil suction is a quantity that can be used to characterize the effect of moisture on volume and strength and, therefore, to determine the physical behavior of soil.[47] It is a measure of the energy[48] that holds the soil water in the pores or a measure of the pulling force exerted on the pore water. Characterizing swell behavior from soil suction tests, as described in Appendix A, is analogous to the procedure for characterizing swell from consolidometer swell tests.

Strength tests

27. Strength tests are required to estimate the bearing capacity of foundation soils at the final or equilibrium water content. A measure of shear strength with depth is also needed to evaluate soil support from adhesion along the shaft of pier foundations. Bearing capacity,

however, is usually not a problem in swelling soil because footings are often placed at depths below the active zone where moisture conditions are not expected to change and bearing pressures are usually less than the swelling pressure.

28. The most common strength tests performed on undisturbed specimens are unconfined compression, unconsolidated-undrained (Q), consolidated-undrained (R), and the drained (S) direct shear.[42] The unconfined compression test may indicate strengths that are too low because the effect of confinement is not considered. The Q and R tests should be performed at confining pressures equal to the calculated in situ overburden pressure. The Q and R tests are considered appropriate because rapid shear associated with failure allows little time for drainage in the relatively low permeable swelling soils. Analyses using total stresses are also often preferred because problems in determining pore and lateral pressures are avoided. The lower limit in the scatter of results of undrained triaxial tests has been recommended when estimating in situ shear strength of stiff fissured clays.[49] The mean undrained strength may be used when scatter is small.

Movement Analyses

29. Analyses of foundation movement are necessary to design a structure that can accommodate the predicted movement without undue distress. Table 1 illustrates important factors that influence the magnitude and rate of foundation movement. The difficulty of predicting potential heave is complicated further by the effect of the type of foundation, depth of foundation, and load exerted by the footings on swelling of expansive soil. Additional problems include estimating the location and amount of available moisture and the final or equilibrium moisture profile.

30. Accurate heave predictions are fortunately not always necessary to determine a rational foundation design. Heave predictions within 20-50 percent have usually been adequate.[50] Observations of existing structures or use of empirical methods can also give a good

first estimate of the probable magnitude of heave. Heave predictions may be needed for pile or pier foundations extending below the active zone to aid estimates of upward drag on portions of the pier within the zone of moisture change.

31. Lateral movement may also affect the integrity of the structure. Lateral thrust of expansive soil with a horizontal force up to the passive earth pressure can cause bulging and fracture of basement walls. Structures constructed on slopes that contain swelling soil may experience some lateral movement as the soil creeps downhill. Seasonal downhill creep is characterized by a slow movement of the soil from cyclic expansion and shrinkage aided by gravity.[51] Creep displacements of 0.4 in./year (1 cm/year) were observed on an undisturbed slope of 12-14 percent (1 vertical on 7 horizontal) in an expansive silty clay soil 5 ft (1.5 m) thick near Stanford University in central California.[52]

Prediction of potential total heave

32. The proportion of volumetric swell that occurs as vertical heave depends primarily on the soil fabric. Vertical heave of intact soil with few fissures may equal all of the volumetric swell, while vertical heave of heavily fissured soil may be as low as one third of the volumetric swell.[8,53] The following methods for predicting potential total vertical heave assume that all of the volumetric swell occurs in the vertical direction. Predictions of lateral movement are beyond the scope of this report.[54,55]

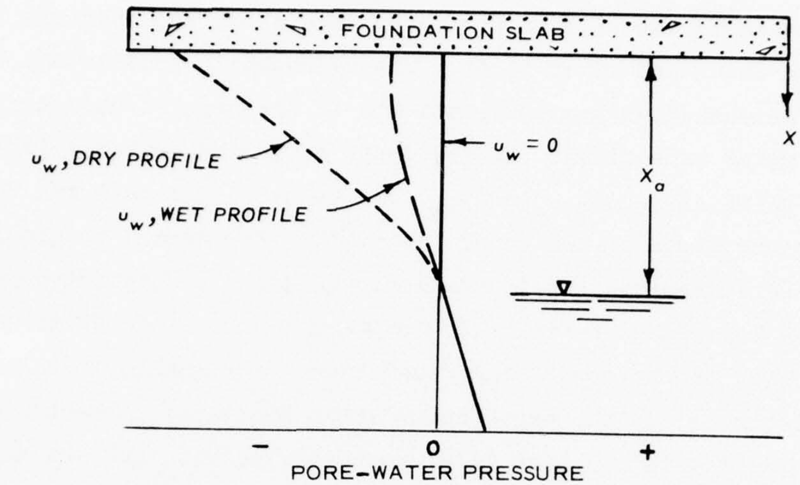
33. Most methods of predicting potential total heave beneath a covered area assume the following final or equilibrium pore-water pressure profiles[8,19,23,42,56] illustrated in Figure 4:

$$\text{Saturated:} \quad u_w = 0 \tag{1}$$

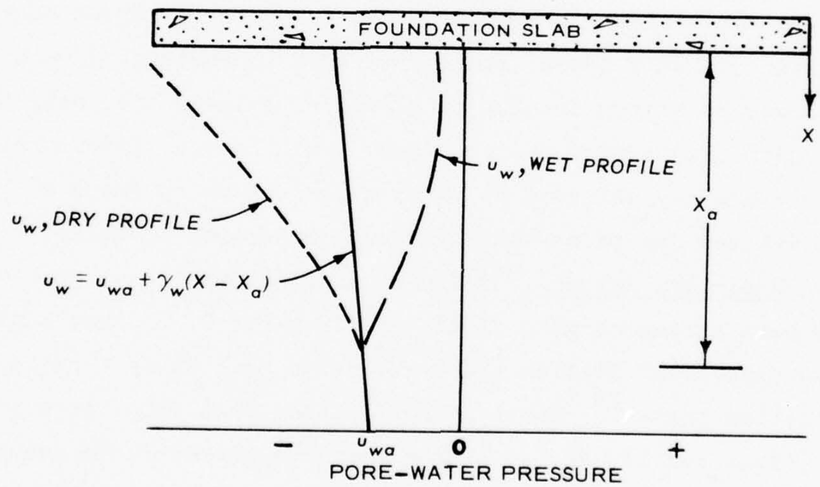
$$\text{Hydrostatic:} \quad u_w = u_{wa} + \gamma_w(X - X_a) \tag{2}$$

where

u_w = pore-water pressure at depth X , tsf



a. Saturated profile



b. Hydrostatic profile

Figure 4. Assumed equilibrium pore water pressure profiles beneath foundation slabs

u_{wa} = pore-water pressure at depth of the active zone X_a, tsf

γ_w = unit weight of water, tons/ft^3

The saturated profile may be more realistic beneath residences and buildings exposed to watering of perimeter vegetation and possible leaking underground water and sewer lines. The hydrostatic profile may be more realistic beneath highways and pavements if drainage is good and ponding of surface water is avoided. If the depth to the water table is less than 20 ft (6 m) in clay soil, then u_{wa} can be set equal to 0 and X_a becomes equal to the depth of the water table.[8,56] For depths to groundwater exceeding 20 ft beneath the foundation, the depth of the active zone can sometimes be assumed between 10 (for moist profiles) to 20 ft (for dry profiles) below the bottom of the foundation. For shallow foundations, X_a can be estimated as the depth below which the water content/plastic limit or soil suction is constant (i.e., not varying with the season).

34. Predictions of seasonal variations in heave from changes in moisture between extreme wet and dry moisture conditions, Figure 4b, are appropriate for perimeter regions of the foundation. These edge effects are important in many cases; e.g., a structure constructed on a wet site followed by a long drought or growth of a large tree near the structure leads to downwarping at the edges. Calculation of seasonal heave between wet and dry extremes requires a measure or estimate of both seasonal wet and dry pore-water pressure or suction profiles.

35. Empirical methods. Table 2 describes empirical methods that gave the best agreement with field data from the U. S. Army Engineer Waterways Experiment Station (WES) expansive soil study from results of classification tests.[19] These methods assume that final pore pressures are zero (Equation 1), an assumption that should result in generally maximum predictions of potential heave from a given initial condition. The volumetric swell from McDowell's method[57] correlated better with field measurements of vertical heave of the WES study[19] and should be used instead of the potential vertical rise (PVR) or one third of the volumetric swell. Both McDowell and McKeen[58] methods require graphs. Van Der Merwe,[60] McKeen, and Johnson[19] methods tend to give maximum values of heave, whereas the remaining methods tend to give minimum

levels expected at the ground surface.[19] These methods have not been checked for limits of applicability. Cross checks of calculations of potential heave from several of these methods may provide a reasonable, but rough estimate of the range of potential heave expected at the ground surface.

36. Snethen, Johnson, and Patrick[62] rated the one-dimensional consolidometer swell from natural water content to saturation ($u_w = 0$) at the in situ overburden pressure of 20 undisturbed clays and clay shales, Table 3. These ratings compared reasonably well with heaves measured at the WES field test sections.[19,63] The classifications may be used without knowing the natural soil suction τ_{nat}, but accuracy and conservatism of the system are reduced. Consolidometer or soil suction tests should be performed on soils that classify as marginal or high. Soils that rate low may not need additional tests, particularly if the liquid limit is less than 40 percent and the plasticity index is less than 15 percent.

37. Parker, Amos, and Kaster[64] rated the potential volumetric swell from wetting at $u_w = -15$ atm (1440 kPa) to $-1/3$ atm (32 kPa) of B2 horizon soils compacted at a confining pressure of 0.25 psi (1.72 kPa). The ratings of the compacted soil lead to very much larger predicted volume changes than the ratings of the undisturbed soil, Table 3. Materials that are not particularly expansive in the undisturbed state could therefore be used as backfill with unsatisfactory results. However, remolding and compacting heavily fissured soil may significantly decrease the mass permeability and reduce penetration of surface moisture into the backfill, leading to less heave, particularly if the backfill is well drained.

38. <u>Consolidometer and soil suction methods.</u> A simple hand method of predicting potential total vertical heave from consolidometer swell tests, assuming a saturated equilibrium moisture profile, is given in Technical Manual TM 5-818-1.[44] Predictions of potential total heave or settlement can also be made from computer programs such as ULTRAT.[19] This program considers effects of loading and soil overburden pressures on volume changes, heterogeneous soils, and saturated or hydrostatic

equilibrium moisture profiles (Equations 1 or 2). Input data includes results of either consolidometer swell or soil suction tests for each stratum.

39. Seasonal heave between extreme wet and dry moisture profiles can be estimated from ULTRAT by taking the difference between heaves computed for both extreme wet and dry profiles, Figure 4a, or sum of the settlement for the wet profile and heave of the dry profile, Figure 4b. It should be noted from Figure 4b that perimeter movement from climatic changes can exceed the long-term heave beneath the center of a covered area.

Prediction of potential differential heave

40. Differential heave results from edge effects for a finite covered area, drainage patterns, lateral variations in thickness of the expansive foundation soil, and effects of occupancy. Examples of effects of occupancy include broken or leaking water and sewer lines, watering of vegetation, and ponding adjacent to the structure. Other causes of differential heave include differences in loading pressure and size of footings.

41. Reliable predictions of actual potential differential heave are probably not possible because of too many unforeseen variables, including future availability of moisture from the climate and effects of human occupancy. Empirical estimates of potential differential heave sometimes assume one half of the total potential heave.[16,65,66] Differential heave up to three quarters of the potential total vertical heave has been measured,[18,19,65] but can vary from zero to as much as the total heave. Differential heave is often the total heave for structures supported on isolated spot footings or drilled piers and will likely approach the total heave eventually for most practical cases.

Prediction of heave with time

42. Heave with time is nearly impossible to predict for each individual case because the location and time when water is available to the soil cannot be foreseen. Local experience had shown that most heave occurs within 5 to 8 years following construction.[15,16,19] If

predictions of heave with time must be made, an analysis[19] shows that diffusion flow can be approximated by an equation similar to the Terzaghi consolidation equation assuming single drainage at the base of the foundation and a triangular stress distribution:[67]

$$t = \frac{0.9F^3 X_a^{\ 2}}{c_{vs}} \tag{3}$$

where

t = time, days

F = fraction of potential heave at time t

X_a = depth of the active zone, ft

c_{vs} = average effective coefficient of swell, ft^2/day

43. Time for heave is given in terms of the average effective coefficient of permeability in saturated soil k_s (ft/day) by[19]

$$t = \frac{0.0086F^3 X_a^{\ 1.73}}{k_s} \tag{4}$$

Coefficients c_{vs} and k_s include the effect of the actual availability of water, whether intermittent or ponded, and are therefore usually not known. Effective coefficients of swell c_{vs} and permeability k_s from results of covered areas on Yazoo, Upper Midway, and Pierre shale were all on the order of 0.02 ft^2/day (2×10^{-4} cm^2/sec) and 0.001 ft/day (4×10^{-8} cm/sec), respectively.[19]

PART III: TOPOGRAPHY AND LANDSCAPING

44. Topography and landscaping may affect surface and subsurface drainage. Both vertical heave of foundation soil and lateral foundation movement from downhill creep of soil on even fairly flat slopes (1 vertical to 7 horizontal)[68] can be aggravated by inadequate drainage and ponding of surface water. Grading and drainage should be provided to drain all surface water away from the structure. Trees should be located a distance away from the structure of about 1 to 1-1/2 times the height of the mature tree.[27,69] The foundation soil may also be treated to reduce the effects of swelling clays and minimize migration of moisture into the soil. Construction in fresh excavations, without replacement of a surcharge pressure equal to the original soil overburden pressure, should be avoided where possible because the reduction in effective stress leads to rebound and heave.

Drainage Techniques

45. Sloping the ground away from the structure will prevent undesirable accumulation of surface water. Drain trenches constructed around the perimeter of the foundation, Figures 5 and 6, can help minimize accumulation of moisture[21,70] and reduce seasonal edge movements. Drains should be placed in catch areas that are likely to collect ponded water.[5] Subsurface interceptor drains should be installed when wetting of foundation soil may occur from gravity flow of free water in subsurface pervious soil layers. Interceptor drains are also effective along the toe of slopes to improve slope stability and prevent landslides.[71] Subsurface drains around the perimeter of swimming pools are also helpful for stabilizing soil moisture.[68,72]

46. Drains should be constructed with watertight and flexible joints and should preferably not be placed in highly desiccated soil. Impervious moisture barriers should be placed beneath the drains because drains and culverts can be sources of water to foundation soil. Typical examples of successful swimming pool construction include a

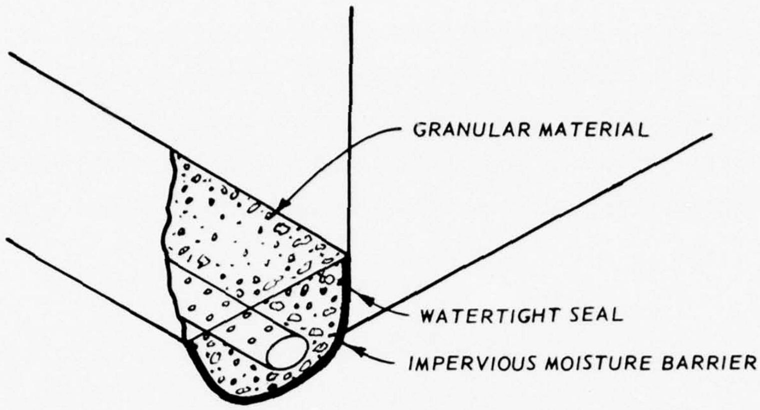


Figure 5. Drainage trench

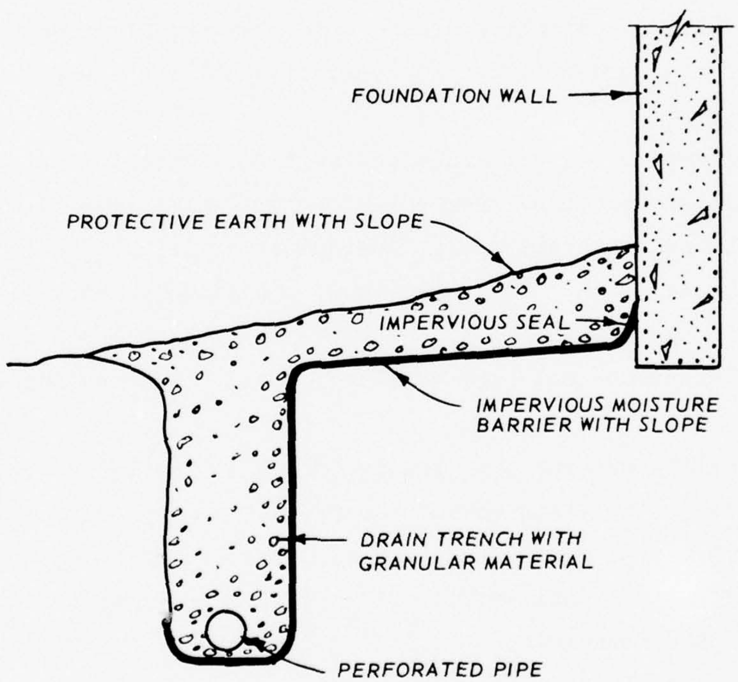


Figure 6. Vertical and horizontal moisture barriers

pervious sand-gravel and subdrain system constructed between the pool and an impervious membrane.[68,73]

47. Drains should collect all water from downspouts, external faucets, and other runoff and carry all surface water away from the structure. Sewer and water lines near the structure should be constructed with watertight and flexible joints and located in foundation soil with least potential for swell, where feasible. All connections with the structure should also be watertight and provided with flexible joints.

Stabilization Techniques

48. The choice of stabilization techniques depends on the economy of the technique, availability of materials and construction equipment, and applicability to the construction site. The most common and successful methods include compaction control (removal and replacement of soil), moisture barriers, prewetting, and chemical stabilization with lime.

Compaction control

49. Compaction control minimizes swell of compacted subgrade soil and backfilled excavations. Removal of about 4-8 ft (1-3 m) of surface swelling soil and replacement with nonexpansive, impervious backfill also helps reduce heave.[1,2,71] Pervious, nonexpansive backfill equipped with drains to carry off infiltrated water can also be used with care.[72,74] Impervious moisture barriers should be placed beneath the drains.

50. Swelling pressures on foundation walls can be reduced to within safe limits by placement of impervious, nonexpansive backfill. Nonexpansive material minimizes the forces exerted on walls, while impervious backfill prevents infiltration of surface water through the backfill into the foundation soil.[71,75] Impervious, nonexpansive backfills can also be placed on level areas to raise the elevation of the foundation and improve drainage from the structure.

51. The potential heave of expansive soil can be reduced by compacting to low density at high water content. Dry density-water content

relationships including superimposed plots of strength or swelling relationships can be developed from laboratory data.[76-78] Graphs similar to Figure 7 can help determine the optimum compaction density and water content to minimize swell.[79] However, controlling volume change potential by compacting at low densities and high water contents may be difficult. An examination of Figure 7 shows that material from Fort Sam Houston has an expansion pressure equal to 1.5 tsf (144 kPa) at 90 percent of optimum dry density (100 pcf) and +5 percent optimum water content (21 percent). A swell of approximately 10 percent under a load of 0.75 tsf (72 kPa) can be computed. The soil will also probably be too wet to work in the field at this water content. Consequently, only replacement with nonexpansive soil or lime stabilization are proven treatments for fill in expansive soil areas. Kneading compaction reduces heave on wetting compared with static compaction.[80] Settlement should be checked if the fill supports foundation footings.

Moisture barriers

52. Perimeter barriers. Moisture barriers or impervious membranes 8 ft (2.5 m) or more in width placed around the perimeter of structures and on shoulders of roads have effectively reduced variations in moisture changes and reduced differential heave.[6,8,21,28,35,36,73,74,81] Soil moisture will probably continue to increase, although more uniformly, beneath the membrane. For example, impervious membranes are not effective in controlling the swell of soil from capillary rise or from a rising water table. Membranes could be detrimental to the performance of some foundations where perimeter backfill soils are more pervious and expansive than undisturbed soil beneath the foundation. Trees, shrubs, and all deep-rooted vegetation should be planted beyond the outer perimeter of the membrane.

53. Membranes are usually made of impervious plastic materials such as polyvinyl chloride (PVC), polyethylene, asphaltic fiberglass sheets, concrete, catalytically blown asphalt, or 3/8-in. (10 mm) sprayed bitumen. Seams, overlaps, and punctures in plastic membranes should be completely sealed to provide an effective vapor barrier. The joint between the membrane and foundation should be impervious. The membrane

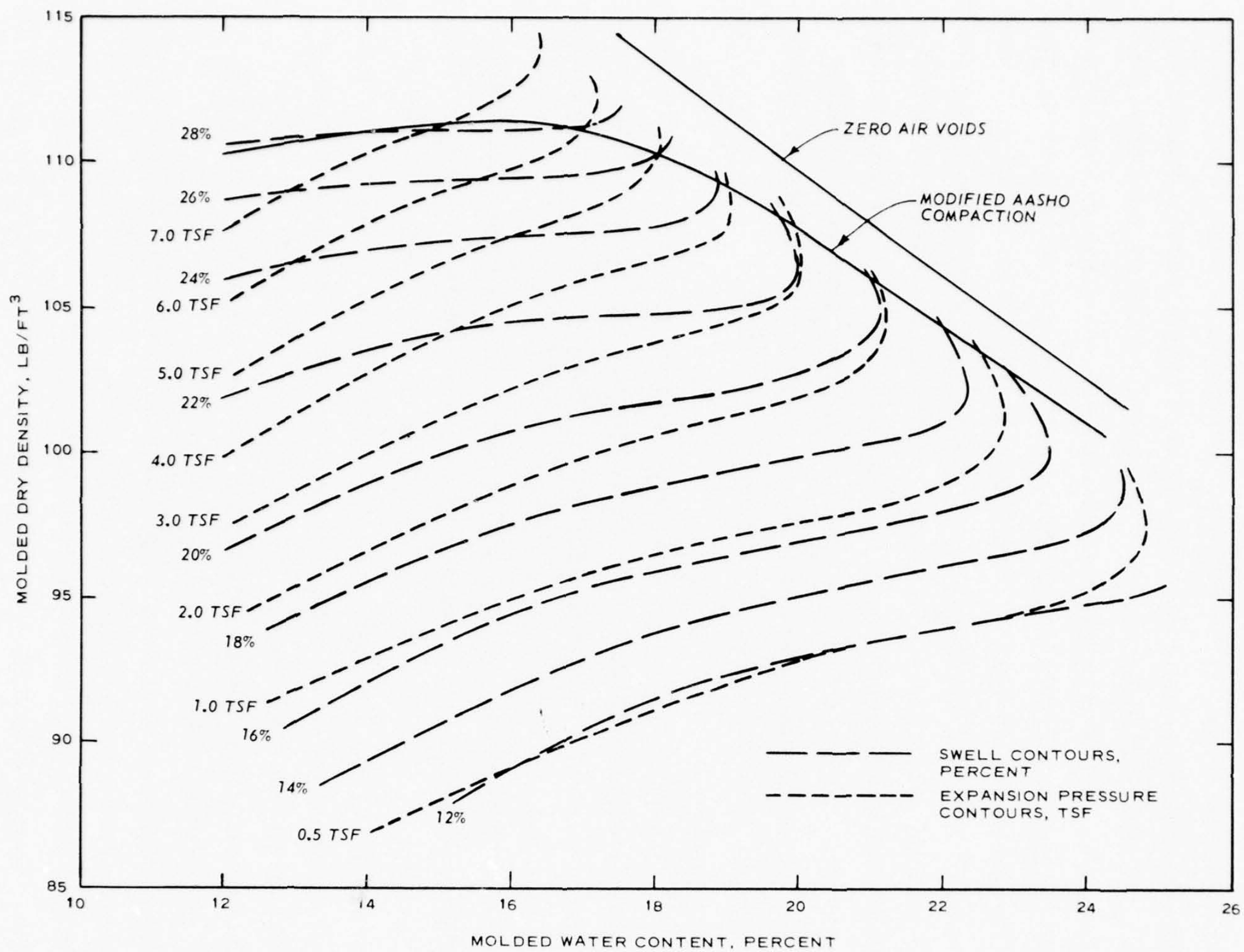


Figure 7. Compaction-swell curves of Fort Sam Houston subgrade material (redrawn from Plate 3, Reference 79)

seal at the foundation should also be flexible to allow some movement, perhaps by placing folds in the membrane.

54. Vertical membranes around the perimeter, Figure 6, are useful in minimizing seasonal edge movements, although moisture may build up beneath the foundation from capillary rise or migration of moisture beneath the bottom edge of the membrane. The vertical barrier is placed about 3 ft (1 m) from the foundation to simplify construction and avoid disturbance of foundation soil. The depth of vertical barrier should extend to the bottom of the active zone of moisture changes. Plastic horizontal membranes should be protected by a layer of earth and care should be taken during placement and when vegetation is planted around the structure to avoid puncturing the membrane.[72]

55. <u>Area barriers.</u> Impervious vapor barriers are sometimes placed beneath concrete slabs or on the ground surface in ventilated crawlways. Vapor barriers beneath the concrete slab in heated areas such as furnace rooms should help minimize loss of moisture from the foundation soil due to the higher vapor pressures in the soil associated with small increases in temperature. The vapor barrier also helps retain moisture in the concrete needed for cure; excess water not needed for cure should be avoided. An impervious membrane on the ground surface in a crawl space may help reduce shrinkage in clayey foundation soils where the water table is deep. Settlement of foundation soils often occurs because the ventilated crawl space prevents precipitation from entering the soil under the house, although moisture continues to evaporate from the soil.[82] A vapor barrier, however, should not be placed on a substantial layer of permeable top soil where a shallow water table exists or site drainage is such that drying is not significant; otherwise, heave may be aggravated.

56. Moisture and insulation barriers help minimize differential heave from thermal effects due to temperature gradients and freezing soil.[81,83] Steep thermal gradients, particularly in cold areas, cause horizontal migration of moisture from hot to cold areas. In Canada, a 2-in.- (5-cm-) thick polystyrene insulative horizontal moisture barrier around the perimeter of the external walls eliminated cold spots and

transfer of moisture from the foundation soil to the outer perimeter and minimized movements between the foundation soil and buildings.[83] Insulation minimizes temperature gradients beneath the perimeter, thereby reducing horizontal diffusion of moisture. Insulation also protects from freezing, which can cause settlement and heave following thaw in swelling soils. This mechanism is opposed to that of frost heave, which occurs from formation of ice lenses in silty soils and lean clays.

57. Moisture can accumulate beneath asphaltic pavements from temperature gradients and can lead to pavement heave.[84] The dark pavement cools by long wavelength radiation at night to temperatures less than at the shoulders. Moisture tends to move laterally from the edges toward the center of the pavement and may also seep through the top seal. Some moisture may diffuse vertically downward during the day, but not enough to prevent accumulation beneath the pavement without special design provisions. Placement of reflective materials on the surface, such as reflective aggregates, zinc oxide or white lead paints, and a layer of thermal insulation beneath the pavement, should reduce long wavelength radiation and minimize temperature gradients. Vertical moisture barriers at the shoulders should aid in minimizing heave from horizontal diffusion of moisture.

58. Moisture barriers can also be useful in minimizing foundation soil heave from chemical reactions between sulfate and carbonates in the soil and water and oxygen diffusing from external sources into the soil.[10,11] A coating of bitumen has given satisfactory protection in an excavation near Lake Erie.[12] Since vapor barriers beneath concrete slabs tend to eliminate the transmission of moisture from soil through the slab, deposition of dissolved sulfates in the concrete from soils containing sulfates should also be minimized, thus protecting concrete slabs from sulfate attack.

<u>Prewetting</u>

59. Prewetting by ponding or submerging an area in water allows desiccated foundation soil to swell and reach a more nearly equilibrium water content prior to construction. Prewetting can be effective, but may require many months unless the foundation soil contains an extensive

fissure system. Prewetting about 2-3 percent above the plastic limit has provided significant improvement of the performance of slab-on-ground foundations.[85,86] Excessive prewetting, however, has been detrimental to foundations where moisture in wetted soil can migrate down into dry deeper soil and cause very high swells.

60. Installation of a grid of vertical sand wells prior to flooding can reduce the time needed for ponding to within a few months.[87] Lime mixed with the ponded water helps to increase the migration of water, apparently through an increase in soil permeability.[2,88] Lime mixed with the top clay layer following ponding can reduce plasticity and increase its firmness as a working platform.[87]

Lime treatment

61. Lime continues to be the most widely used and most effective additive for stabilization of expansive clays, although lime treatment is not always successful.[2] Lime stabilization develops primarily from base exchange and cementation processes. During base exchange, the positive Ca^{++} ions from the lime are adsorbed by the clay particles, displacing some Na^{+} ions, as a result of the negative surface charge of the clay particles.[89] The ions become hydrated and restrict water adsorption on the particle surfaces. The +2 valence limits the distance of penetration of the negative charge from the clay particles into the pore water. Cementation is a long-term chemical or pozzolanic reaction in which lime reacts with clay mineral constituents to form compounds such as calcium silicate hydrates and calcium aluminum hydrates that probably interlock with the clay particles to form permanent bonds.

62. Small additions of lime from 2-8 percent usually decrease the plasticity index and swell and increase the permeability and shear strength of expansive clays.[1,2,88] In some cases, lime may worsen the swelling characteristics, depending on the structure and composition of the expansive soil.[88] Additions of 2-6 percent cement with lime should further improve the effectiveness of lime treatment.[1,71] Cement stabilization alone is usually adequate with some kaolinitic and illitic soils.

63. The effectiveness of lime treatment depends on the thoroughness of mixing.[90] Pressure injection of lime may be effective in soils

containing extensive fissures and cracks into which the slurry can be injected.[84,91] The injected slurry deposited in fissures appears to provide an effective lime barrier against moisture flow as well as pre-wet the soil from sorption of the slurry.

PART IV: SELECTION OF THE SUPERSTRUCTURE AND FOUNDATION

64. The design of the superstructure and foundation should be chosen to satisfy most economically the functional requirements of the structure, minimize soil differential movement, and minimize damages that may occur to the structure from soil movement. The functional requirements may require, for example, a structure that will limit the deflection/length ratio to less than a certain amount. The foundation should be designed to transmit no more than the maximum tolerable distortion to the superstructure, as demanded by usage requirements, avoiding excessive overdesign. The superstructure should tolerate movements transmitted by the foundation such that the structure continues to contribute aesthetically to the environment and maintenance will remain on a minor level.

65. Table 4 illustrates the interrelationship of various foundation and superstructure systems that may be designed to minimize or resist the predicted differential heave avoiding unacceptable structural distress. The predicted differential heaves, Table 4, refer to heave beneath lightly loaded, flexible covered areas. Stiffening beams significantly reduces the differential distortion of concrete slabs. A beam-on-pier foundation will tend to eliminate effects of heaving; however, possible soil movement beneath the footings of deep foundations such as piers should be checked.

Superstructure Systems

66. The superstructure should flex or deform compatibly with the foundation. Frame construction, open floor plans, and truss roofs tend to minimize damages from differential movements.[100] The choice of the type of first floor, frame, and wall should depend on the choice of foundation. Table 5 describes the various superstructure systems given in Table 4.

First floor

67. The design of the first floor should be selected to maintain differential movements within permissible limits. Certain types of structures, such as warehouses, shops, and hangers with few internal walls and partitions, can tolerate fairly large differential heaves such that a slab-on-grade isolated from exterior walls may be sufficient. Brick walls, Table 5, can tolerate larger deflection/length ratios than 1/500 if the rate of distortion is sufficiently slow.[85] Interior walls, partitions, doors, and service equipment should be designed to tolerate the anticipated floor movements. Reinforced and stiffened mat slabs or suspended first floors on grade beams and piers may be necessary to minimize differential heave to within acceptable levels in residences and single or multi-story buildings.

Frames

68. The frame should be selected to tolerate the maximum differential movement transmitted by the foundation. The type of framing system should not ordinarily be limited with properly designed shallow, continuous footings and beam-on-pier foundations. Shallow footings should be placed in sands, gravels, or soils with low potential heave. Beam-on-pier foundations can avoid effects of swelling soil by passing the shafts through the unstable strata. In some cases, footings are required to be placed in nonideal locations where swell or consolidation beneath the footings may present a problem. The frame should then be sufficiently flexible to tolerate the anticipated differential movement between footings. Frames can be fairly easily adapted to accommodate the deflection of mat slabs, which can be designed to permit various amounts of distortion. Reinforced and stiffened mat slabs are usually designed not to exceed a deflection/length ratio of 1/500.[53,85,92,96]

Walls

69. Walls should tolerate the maximum differential movement transmitted by the foundation and framing system. Cracks detract aesthetically from the appearance of the structure, weaken structural walls, and reduce insulation from the outside environment. Control joints may be used to increase flexibility of rigid or semirigid walls. Walls can

be attached to the framing system with flexible connections. Examples of frame and wall construction are given in References 7 and 71.

Foundation Systems

70. Various possible foundation systems that are consistent with the functional and architectural requirements of the total structure and adaptable to the local topography and subsurface features should be compared to determine relative performance. Optimum performance can be described as the ability to minimize or resist the maximum anticipated differential movement to within acceptable limits while providing the most economy. Appendix B describes remedial measures for foundations that have not been adequately designed and originally provided with adequate landscaping or soil stabilization.[16,71,105,106]

Shallow individual and continuous footings

71. Structures supported by shallow individual or continuous wall footings are susceptible to damages from lateral and vertical movement of foundation soil, Table 4. Dishing or substantial settlement may occur in clays, especially in initially wet soil, where a well ventilated crawl space is constructed under the floor.[82] The crawl space prevents precipitation from entering the soil, but evaporation of moisture from the soil continues. Center heave, Figure 1, can occur if the top layer of soil is permeable and site drainage is poor. Damages from differential heave or settlement include door jamming, cracking of internal partitions, and separation of internal partitions from the floor and roof.[82] Fractures[24] may appear in walls after deflection/length ratios exceed about 1/1000 or differential movement exceeds about 0.5 in. (13 mm).[102]

72. Shallow footings may be used where expansive strata are sufficiently thin to locate the footing in a nonexpansive stratum below which differential movement is negligible. Placing heavy loads on these footings may not be effective in countering high swell pressures because of the relative small width of the footings.[70] The stress imposed on

the soil is very low below a depth of about twice the footing width and contributes little to counter the swell pressure unless the expansive soil layer is thin.

73. Basement walls of reinforced concrete can be constructed directly on the foundation soil provided foundation pressures are less than the allowable bearing capacity.[71] Steel reinforcement can provide the necessary restraint to horizontal earth pressures. Unreinforced masonry brick and concrete blocks should not be used to construct basement walls.

Reinforced mat slab

74. The reinforced mat slab is often suitable for small and lightly loaded structures, especially if the expansive or unstable soil extends nearly continuously from the ground surface to depths that exclude economical deep pier foundations. The mat slab has been found more economical in Australia for placement on uncontrolled fills than pier and beam foundations.[82] A thick reinforced mat is suitable for large, heavy structures. The rigidity of these mats minimizes distortion of the superstructure from both horizontal and vertical movements of the foundation soil.[5] Increasing the stiffness of the slab and superstructure also reduces differential heave. Supporting pressures beneath stiffened slabs can become very nonuniform and cause localized consolidation of the foundation soil. Concrete slabs without internal stiffening beams are much more susceptible to doming from heaving soil. Edge stiffening beams beneath reinforced concrete slabs have prevented significant moisture loss and have reduced differential movements beneath the slab.[21,25,85]

75. The reinforced waffle concrete mat usually consists of a 4-in.- (100-mm-) thick slab stiffened with underlying crossbeams, Figure 8. The 4-in. slab transmits the loading forces to the beams, which resist the moments and shears due to differential heave of the expansive soil. Beam spacings should be limited to 20 ft (6 m) or less. Beam widths should be 8-12 in. (200-300 mm).[53,92,96-98] Construction joints should be placed at intervals of less than 150 ft (45 m) and cold joints less than 65 ft (20 m).[96] Concrete strength should be

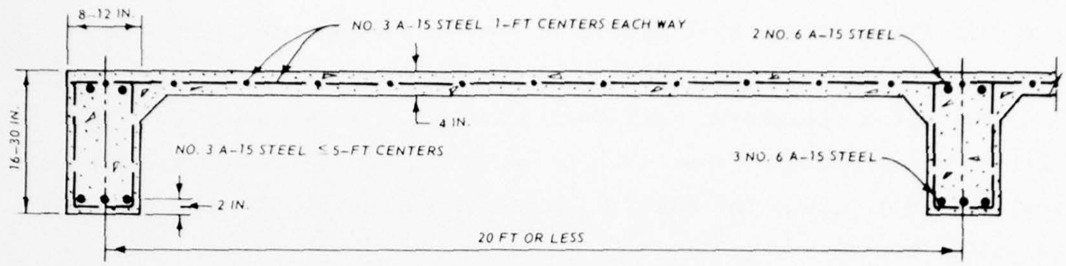


Figure 8. Reinforced waffle mat slab

3000 psi (20.7 MN/m^2) with about 0.5 percent reinforcing steel. The mat may be inverted (stiffening beams on top of the slab) in cases where bearing capacity of the surface soil is inadequate or a supported first floor is required.[5]

76. <u>Support index.</u> Table 6, reprinted from Holland et al.,[107] compares four rational methods that have provided successful designs of reinforced waffle slabs. All of these methods have in common a support index C, the ratio of area supported by the foundation soil to the total area of the slab, or a similar parameter denoted as the edge lift-off distance e. The edge lift-off distance may be directly related to the support index.[24,85] The significant limitation of all of these methods is that reasonable values for the support index C or edge lift-off distance e may be difficult to evaluate.

77. The Building Research Advisory Board (BRAB)[97] method evaluates the support index as a function of a climatic rating system and plasticity indices. This method is too conservative in some cases, particularly for long slabs greater than 6β [24,25,85,109]

$$\beta = \sqrt[4]{\frac{EI}{E_s}} \tag{5}$$

where

β = relative stiffness length, ft

E = creep modulus of elasticity of concrete, tsf

I = gross moment of inertia of the slab section, ft^4

E_s = modulus of elasticity of soil, tsf

The main fault of the BRAB method is that the moment of inertia of a cracked beam section is assumed.[107] The BRAB support index also ignores many important parameters that should influence a realistic C, such as initial soil moisture, availability of water, and thickness and type of swelling soil; i.e., the BRAB C does not adequately account for differential heave.

78. The methods of Lytton,[94] Walsh,[95] and Fraser and Wardle,[108] are essentially extensions of the BRAB method and attempt to determine a more rational support index. These latter attempts result in the need to determine the mound shape of the expansive soil beneath the slab defined in terms of the edge lift-off distance e and maximum differential swell y_m. However, the mound shape is as difficult to determine for practical design cases as is the BRAB support index. The BRAB, Lytton, and Walsh methods can be fairly easily applied to the design of reinforced mat slabs after the support index or mound shape is determined. Both Lytton and Walsh design methods gave closest agreement with field data, bracketing measured field deflection/length ratios for construction on an initially dry site in Australia.[107]

79. The Post-Tensioning Institute (PTI)[110] has developed a new, but untested, design procedure with the intent to improve the rational basis for determining the mound shape and efficiency of design. This method shows that the edge lift-off distance is similar to the relative stiffness length β and that all maximum differential slab deflections occur within a distance of 6β for slabs longer than 6β. Maximum shear was developed at or near the perimeter of the slab and within one β length of the perimeter.

80. Preliminary design. Three designs for reinforced waffle slabs described in Table 4 differ in the beam depth and spacing, depending on the predicted maximum differential heave and effective plasticity index. The deeper beam depths and smaller beam spacings for each of the light, medium, and heavy slabs, Table 4, tend to provide conservative designs. These designs are conservative in view of still undetermined fully acceptable or finalized uniform design criteria and relatively high repair cost of fractured reinforced and stiffened slabs. The heaviest

mats with 30-in.- (750-mm-) deep beams were observed to do well in high movement areas, such as San Antonio, Tex., Montgomery, Ala., and other locations.[111]

81. Modifications to the three types of standard mats in Table 4 can be made during the detailed engineering design phase using conventional practice[92,96] or the new PTI procedure[110] to help ensure adequate resistance to moment, deflection, and shear resulting from structural loading forces and to minimize overdesign. Beam spacings should be adjusted to support column, wall, or concentrated loads. The slabs are usually designed for deflection/length ratios of 1/480. The PTI procedure designs the slab for a deflection/length ratio of 1/480 with center lift and 1/800 with edge lift.

82. Post-tensioned reinforced mat slabs may be slightly stronger than an equivalent section of a conventionally reinforced mat slab, but trained personnel and careful inspection are required to properly apply the post-tensioning procedure. Tendons should be stressed 3-10 days following the concrete pour such that the minimum compressive stress in the concrete exceeds 50 psi (345 kPa). Stressing within the 10-day limit eliminates much of the shrinkage cracking. Stressing should also be completed before structural loads are applied to the slabs.

83. Placement of a pervious 6-in. (15-cm) granular layer on top of the original ground surface before construction of the slab may help reduce differential heave due to the additional surcharge load. The granular layer on top of the original ground surface also helps to provide a slope leading down and away from the structure, improving drainage and minimizing the possibility that the granular layer could provide a source of moisture to desiccated foundation soils. Drainage and soil stabilization techniques for minimizing differential heave described in Part III should be used with slab foundations to increase the performance of reinforced mat slabs.

Beam-on-drilled pier

84. The drilled pier foundation provides an economical method for transfer of structural loads from unstable (weak, expansive) to deeper stable (firm, incompressible) strata, and it is generally more

economical than other forms of piling if the hole can be bored.[112] Occasionally when the firm bearing stratum is too deep for the shaft to bear directly on a stable stratum, the drilled pier is designed as a friction or floating shaft, securing its support entirely from adhesion with the surrounding clay. Detailed applications including advantages and disadvantages of drilled pier foundations are described in Table 7. The pier foundation may be economical compared with traditional strip footings,[99,114] particularly in open construction areas and with pier lengths less than 10-13 ft (3-4 m) or if the active zone is deep, such as areas influenced by tree roots.[105] Beam-on-pier foundations, in fact, have been preferred in the expansive soils of the Denver area rather than reinforced waffle slabs, which have been too uneconomical to construct.[70]

85. The design and construction of beams-on-drilled piers must be closely controlled to avoid failures. Most failures have been caused by defects in construction and by effects of swelling soils, Table 8. Defects attributed to construction techniques include discontinuities in the shaft, caving of soils, and distortion of the steel reinforcement.[113,115] Failures from effects of swelling soils include wetting of subsoils beneath the base,[19,82] uplift,[101] lack of air gap beneath grade beams,[116] and lateral movement from downhill creep of expansive clay.[111] The rise of pier foundations from soils swelling beneath the base has caused many failures.[82,117]

86. Designs of beam-on-pier foundations have usually been based on empirical procedures, limited load test data, and the behavior of existing structures. Consequently, the designer needs much experience and expertise.[118] Designs have usually been satisfactory where subsurface conditions are well established and relatively uniform and the performance of past construction is well documented.[71,115,118] The design of drilled piers should consider bearing capacity, skin resistance, uplift forces, construction techniques, and inspection.

87. <u>Bearing capacity.</u> Shear failure of the bearing stratum and structural loads exceeding the strength of the concrete shaft are normally not problems. Heave or settlement of the foundation usually controls the design and should not exceed specified limits set by usage

requirements and tolerances of the structure. Present theoretical concepts and empirical correlations permit reasonably reliable predictions of ultimate bearing capacity, but not those of heave or settlement. Consequently, factors of safety applied to the ultimate bearing capacity are most commonly used to determine safe working loads. Experience[113,119] shows that working loads of one-half to one-third of the ultimate bearing capacity including skin resistance (factor of safety 2-3) adequately protect against a bearing failure and usually maintain settlements, but not heave, within tolerable limits of about 0.5 in. (13 mm).

88. The load-carrying capacity of a pier depends on both end bearing and skin friction from side shear. The interaction of stresses between end bearing and skin friction is commonly assumed negligible such that the ultimate load Q_o is calculated as the sum[118]

$$Q_o = Q_p + Q_s = q_p A_p + f_s A_s \tag{6}$$

where

Q_p = ultimate base load, tons
Q_s = ultimate shaft load, tons
q_p = ultimate base resistance, tsf
A_p = bearing area of pier base, ft^2
f_s = ultimate shaft resistance, tsf
A_s = perimeter area of pier shaft, ft^2

89. The base resistance q_p is conventionally given as[118]

$$q_p = cN_c + \bar{\sigma}_v N_q \tag{7}$$

where

c = strength intercept (cohesion) of the assumed straight-line Mohr envelope, tsf
N_c, N_q = dimensionless bearing capacity factors evaluated by methods given in Reference 118
$\bar{\sigma}_v$ = effective vertical stress in the ground at the foundation level, tsf

The cohesion c is normally determined from undrained Q or R tests. N_c is approximately 9 in cohesive soil ($\phi = 0$) for depths greater than 4 or 5 shaft diameters. N_q is 1 for cohesive soil and usually ignored, being approximately compensated by the weight of the concrete shaft. Development of full end bearing requires settlements from 10-30 percent of the shaft diameter.[113,118]

90. Skin resistance. Skin resistance develops from small relative displacements between the shaft and adjacent soil. Positive skin friction, which helps to support structural loads, develops when the shaft moves down relative to the soil. Negative skin friction, which adds to the structural loads and increases the end bearing force, develops when the surrounding soil moves down relative to the shaft.[119] The capacity of drilled and underreamed piers cast in expansive soil has generally been designed in the past for end bearing only without side friction.[7] Soil was assumed to shrink away from the sides of the shaft during droughts at the perimeter of covered areas to some depth X_a below the ground surface. Excluding skin friction in the design capacity may be grossly over-conservative for many cases because numerous load tests have shown that a large proportion of the total shaft load is usually taken by positive skin friction. Shrinkage effects have only been observed 1 to 2 shaft diameters below the ground surface.[113,118]

91. The skin friction f_s may be evaluated by the equation[118]

$$f_s = c_a + q_s \tan\psi \tag{8}$$

where

c_a = adhesion, tsf

q_s = normal stress acting on the pier shaft $K\bar{\sigma}_v$, tsf

K = ratio of horizontal to vertical effective stress

$\bar{\sigma}_v$ = vertical effective stress, tsf

ψ = angle of friction between the soil and pier shaft, degrees

The angle ψ is very close to the effective angle of internal friction ϕ' for remolded cohesive soil.[118] Skin resistance is usually fully mobilized with a downward displacement of 0.5 in. (13 mm) or less or

about 2 percent of the shaft diameter.[113,118] These displacements are much less than those required to fully mobilize end bearing.

92. Because observations taken after sufficient time have indicated that skin friction becomes approximately equal to the undrained undisturbed shear strength c_u, skin resistance has been compared with c_u for all clays:[17,118]

$$f_s = c_a = \alpha_f c_u \tag{9}$$

in which α_f is a reduction coefficient that varies between 0.2 and 1.5, depending on type of pier and soil conditions. The reduction coefficient is about 1 for soft clays and decreases as the strength of the clay increases. The average α_f appears to be about 0.5-0.6 for drilled piers in overconsolidated clay.[113,118] An α_f of 0.3 is recommended if little is known about the soil. The reduction factor approaches zero near the top and bottom of the piers, reaching a maximum near the center of the shaft. The reduction of α_f at the top is attributed to soil shrinkage and low lateral pressure, while the reduction at the bottom is attributed to interaction of stress between end bearing and skin resistance. The reduction coefficient does not exceed 1.0 when taken as the ratio of the mobilized shear stress to the actual shear strength of the soil adjacent to the shaft following installation. Construction causes some remolding of adjacent soil, particularly for driven piles.

93. Skin resistance may also be evaluated in terms of effective stress.[118] Experience[7,66,69,120] indicates that skin friction may be calculated from results of drained (S) direct shear tests

$$f_s = c' + K\bar{\sigma}_v \tan \phi' \tag{10}$$

where c' is the effective cohesion and ϕ' is the effective angle of internal friction. Satisfactory values for K were 1-1.5 in swelling cohesive soils for piers subject to uplift forces.[120,121]

94. <u>Uplift forces.</u> Uplift forces, which are a direct function of swelling pressures,[71] will develop against surfaces of pier foundations

when wetting of surrounding expansive soil occurs. Side friction resulting in uplift forces should be assumed to act along the entire depth of the active zone since wetting of swelling soil causes volumetric expansion and increased pressure against the pier shaft. The pier tends to be pulled upward inducing tensile stresses and possible fracture of concrete in the shaft, as well as possible upward displacement of the entire shaft. Moisture may also seep beneath the base of the pier, perhaps by moisture migrating down the soil-pier interface or through the concrete in the pier shaft wetting desiccated swelling soil beneath the base and contributing to the upward displacement.[39,122]

95. The pier foundation should be of proper diameter, length, and underreaming, adequately loaded, and contain sufficient reinforcing steel to avoid both tensile fractures and upward displacement of the shaft. Simply loading a relatively small diameter footing such as a pier, even near the swelling pressure, is not always effective in eliminating swell of expansive soil beneath the base.[70,123] The shaft can sometimes be lengthened to eliminate the need for an enlarged base, particularly in soils where enlarged bases are very difficult to construct.

96. Several rational approaches for estimation of uplift forces in swelling soil are available.[7,71,120,124,125] Appendix C describes a new approach for analysis of uplift forces, including analysis of pier movements and restraining forces. Comparison of limited field data from two instrumented test piers with results of this new approach is considered satisfactory. Empirical equations were derived as an example application of this approach for estimating pier dimensions and required percent reinforcing steel to counter tension forces that may develop in the shaft, provided that the base is placed in nonexpansive or stable soil.

97. The most conservative estimate of pier length needed to prevent pier uplift in a homogeneous soil is to assume undrained strength behavior ($\phi = 0$) and zero loading on the shaft ($P = 0$) based on empirical Equations C7 and C8 of the example analysis in Appendix C:

$$D_p = 1.5 \text{ ft}: \quad L = 2X_a - 1.42\left(\frac{D_b}{D_p}\right)^{2.5} \qquad (11a)$$

$$D_p = 2.5 \text{ ft}: \quad L = 2X_a - 1.76\left(\frac{D_b}{D_p}\right)^{3} \qquad (11b)$$

where

D_p = shaft diameter, ft

L = pier length, ft

D_b = base diameter, ft

If the shafts are straight with no underream ($D_b/D_p = 1$), the length should be twice the depth of the active zone X_a. If the piers are underreamed with $D_b/D_p = 2.5$, X_a can extend down to the base of the footing ($X_a = L$) for pier lengths up to 15 ft (4.5 m) and 25 ft (7.6 m) with diameters of 1.5 ft (0.45 m) and 2.5 ft (0.76 m), respectively, with no danger of uplift from skin friction. These equations are valid for swell pressures exceeding 1 tsf (96 kPa) and soil adhesions c_a less than 1 tsf. Smaller swell pressures increase the conservatism of the above equations.

98. The amount of reinforcing steel must be adequate to take all of the tension stress that is expected to develop in the concrete shaft. The tension force T (a negative quantity) is conventionally estimated by[71,120]

$$T = P - \pi D_p f_s X_a \qquad (12)$$

where P is the loading force. Based on a limited parametric study using the new approach (Appendix C), the percent steel A_s may be estimated by

$$A_s = 0.094 \frac{Lc_a}{D_p} + 0.00275 \frac{L^2 K \tan \phi}{D_p} - 0.03 \frac{P}{D_p^2} \qquad (13)$$

where c_a is the soil adhesion (in tsf) and P is the loading force (in tons). The allowable stress in the steel reinforcing was assumed 60,000 psi (414 MPa) or ASTM A615 Grade 60. Equation 13 shows that the required percent steel is generally larger in smaller diameter piers. The reinforcing steel should be continuous along the full length of the shaft and extend into the underream. Standard hooks are sometimes used in the vertical reinforcing steel of the underream to develop the required bond. The amount of reinforcing is typically 1 percent, but can be as high as 7 percent.[71]

99. Preliminary design. The base of the piers should be located below the depth of the active zone, preferably within a free-water zone or nonexpansive soil to reduce heave beneath the pier. Footings may be placed beneath swelling soil near the top of a granular stratum to avoid "fall-in" of material while underreaming a bell. Standard shaft and bell diameters should be used and variations in pier diameters minimized to simplify construction, reduce contractor equipment on the site, and reduce cost.

100. Underreams are often used to increase anchorage to resist uplift forces. Underreams may be bored in dry or cased holes of at least 1.5 ft (450 mm) diameter and preferably 2.5 ft (767 mm) where inspections are possible to ensure cleanliness of the bottom. The belled diameter should not exceed 3 times the shaft diameter and may be constructed with either 45- or 60-deg bells. The 45-deg bell causes larger stress concentrations than the 60-deg bell, but the 45-deg bell requires less concrete and less cutting time.[126]

101. Straight shafts may be more economical than underreams if the bearing stratum is hard or if subsoils are fissured and friable. Belled piers have not been extensively used in the Denver area because the underream reduces the contact pressure bearing on potentially expansive soil and restricts the minimum diameter that may be used.[71] If bells are not feasible, uplift forces can be controlled by extending the shaft length further into stable, nonswelling soil.

102. Uplift forces may be further controlled by constructing widely spaced piers with small shaft diameters and loading forces

consistent with the soil bearing capacity. Wide spans between piers reduce angular rotation of the structural members. The minimum spacing of piers should be about 12 ft (4 m)[71] or 8 times the shaft diameter[119] to minimize effect of adjacent shafts. Underreamed piers with shaft diameters less than 1 to 1.5 ft (300 to 457 mm) can be difficult to construct. Reese and Wright[113] recommend a minimum diameter of 1.5 ft (457 mm) except for very special circumstances. The upper portion of the pier should be kept vertically plumb (maximum variation of 1 in. in 6 ft (2.5 cm in 1.8 m)) and smooth to reduce adhesion between the swelling soil and pier shaft. Friction reducing material such as roofing felt, bitumen slip layers, PVC, or polyethylene sleeves may also be placed around the upper shaft to reduce both uplift and downdrag forces.[75,105,127] Vermiculite, pea gravel, or other pervious materials should be avoided.

103. Construction techniques. Three methods of drilled pier construction are available: dry, casing, and slurry displacement methods.[113] The dry method is applicable to soil above the water table that does not cave-in or slump when the hole is drilled to its full depth. The casing method is used when excessive caving or slumping occurs in one or more strata. Slurry displacement may be used instead of the casing method and may be preferable for deep caving soils. Care should be exercised to ensure that concrete does not mix with water when placing concrete in areas where groundwater is a problem or when using the slurry displacement method.

104. Concrete strength of at least 3000 psi (20.7 MPa) should be used and poured as soon as possible and on the same day as drilling the hole. Care should be exercised while pouring the concrete to (a) ensure continuity while pulling the casing, (b) ensure tip of tremie is always below the column of freshly poured concrete, and (c) ensure adequate strength of the rebar cage to minimize distortion and buckling. Vibration of concrete helps eliminate voids in the pier.[7] High concrete slumps of 4-6 in. (10-15 cm) and limited aggregate size (one third of the rebar spacing[113]) are recommended to facilitate the flow of concrete through the reinforcement cage and to eliminate cavities in the pier.

105. Mushrooming at the top of the pier from excessive placement of concrete should be avoided.[70,71] The mushroomed area is subject to uplift forces from underlying swelling soil and could cause the pier to uplift. The use of sonotubes or cardboard cylinder forms is one way of eliminating mushrooms.

106. Grade beams. Grade beams spanning between piers are designed to support wall loads imposed vertically downward, but are not designed to resist loads imposed vertically upward on the bottom of the grade beam by heave of expansive soil. Steel is recommended in both the top and the bottom of the grade beam.[7,57] Grade beams are isolated from underlying swelling soil with an air gap of about 6-12 in. (15-30 cm). A convenient method is the use of cardboard forms known as "Verticel," which are wrapped in plastic and will support the concrete, but will deteriorate after the plastic is punctured.[71] The cardboard forms will collapse before swell pressures in underlying soil can deflect or damage the grade beams. Styrofoam forms are not recommended because these may have high crushing pressures and may transmit significant upward pressure to the grade beams.

107. Installation of grade beams and cardboard forms may require overexcavation of soil in the bottom of the grade beam trench between piers. Retainer forms may otherwise be necessary. Interior and exterior walls and concentrated loads should be mounted on the grade beams. Floors may be suspended from grade beams at least 6 in. (15 cm) above the ground surface or placed directly on-grade if the slab is isolated from the walls. Support of grade beams, walls, and suspended floors from sleeper piers or supports other than the pier foundation should be avoided.

108. Inspection. The foundation engineer should visit the construction site during drilling of the first pier holes to verify the foundation design and periodically thereafter to check the need for modifications in the design. The purpose of locating the footings at the selected depth should be emphasized during this first visit and the inspector cautioned to ensure that the intent of the design is accomplished during construction. The structural engineer should also visit

the construction site to emphasize important details of the design to the inspector who otherwise may not rigorously enforce such details.[5,7] Additional details on inspection can be found in Reference 115.

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Table 1

Factors Influencing Magnitude and Rate of Volume Change

Factor	Description
	Soil Properties
Composition of solids	Active clay minerals include montmorillonites and mixed layer combinations of montmorillonites and other clay minerals.
Concentration of pore fluid salts	High concentrations of cations in the pore fluid tend to reduce the magnitude of volume change; swell from osmosis can be significant over long periods of time.
Composition of pore fluid	Prevalence of monovalent cations increase shrink-swell; divalent and trivalent cations inhibit shrink-swell.
Dry density	Larger dry densities cause closer particle spacings and larger swells.
Structure	Flocculated particles tend to swell more than dispersed particles; cemented particles tend to reduce swell; fabrics that slake readily may promote swell.
	Environmental Conditions
Climate	Arid climates promote desiccation, while humid climates promote wet soil profiles.
Groundwater	Fluctuating and shallow water tables provide a source of moisture for heave.
Drainage	Poor surface drainage leads to moisture accumulations or ponding.
Vegetative cover	Trees, shrubs, and grasses are conducive to moisture depletion by transpiration; moisture tends to accumulate beneath areas denuded of vegetation.
Confinement	Larger confining pressures reduce swell; cut areas are more likely to swell; lateral pressures may not equal vertical overburden pressures.
Field permeability	Fissures can significantly increase permeability and promote faster rates of swell.

Table 2

Empirical Methods For Predicting Heave

Reference*	Description**
McDowell, 1959 (57)	A procedure based on swell test results of compacted Texas soils. Field heave estimated from a family of curves using Atterberg limits, initial water content, and surcharge pressures of each soil stratum. The initial water content is compared with maximum (0.47LL+2) and minimum (0.2LL+9) water contents.
Van Der Merwe, 1964 (60)	$S_p = \frac{100}{n} \sum_{H=1}^{H=n} F_H \cdot PE$ in which F_H is a reduction factor to account for pressure at depth H and found from $H = 20 \log F_H$; PE = 1, 1/2, 1/4, 0 in./ft for very high, high, medium, and low degrees of expansion, respectively. The degree of expansion is found from a chart of plasticity index and percent clay fraction. 1 ft = 0.305 m; 1 in. = 25.4 mm.
Vijayvergiya & Ghazzaly, 1973 (61)	$\log S_p = 1/12(0.44LL - w_o + 5.5)$ from initial water content to saturation for 0.1 tsf (10.7 kN/m^2) surcharge pressure.
Schneider & Poor, 1974 (59)	$\log S_p = 0.9(PI/w_o) - 1.19$ for no fill or weight on the swelling soil to saturation.
McKeen, 1977 (58)	A procedure relating soil suction with percent swell including effect of surcharge pressure. Requires use of graphs, shrinkage limit, plasticity index, liquid limit, percent clay fraction, and estimates of initial and final soil suctions.
Johnson, 1978 (19)	$PI \geq 40 \quad S_p = 23.82 + 0.7346PI - 0.1458H - 1.7w_o + 0.0025PIw_o - 0.00884PIH$ $PI \leq 40 \quad S_p = -9.18 + 1.5546PI + 0.08424H + 0.1w_o - 0.0432PIw_o - 0.01215PIH$ for 1 psi (6.9 kN/m^2) surcharge pressure to saturation.

* Superscript numerals and other mentioning of references by number in these tables refer to the similarly numbered sources listed in the References section at the end of the main text.

** S_p = percent swell; LL = liquid limit in percent; PI = plasticity index in percent; w_o = initial water content in percent; H = depth of soil in feet.

Table 3

Relative Swell Between Undisturbed and Compacted Soil

Classification of Potential Swell	Potential Swell S_p, percent	Liquid Limit LL, percent	Plasticity Index PI, percent	Natural Soil Suction τ_{nat}, tsf (kPa)
		Undisturbed Soil[62]		
Low	<0.5	<50	<25	<1.5 (144)
Marginal	0.5-1.5	50-60	25-35	1.5-4.0 (144-383)
High	>1.5	>60	>35	>4.0 (383)

Classification of Potential Swell	PVC,* percent	COLE**	Plasticity Index PI, percent
	Compacted Soil[64]		
Low	<10	<0.03	<10
Medium	10-20	0.03-0.06	10-20
High	20-30	0.06-0.09	20-30
Very high	>30	>0.09	>30

* Potential volumetric swell.

** Coefficient of linear extensibility.

Table 4

Foundation and Superstructure Systems

Predicted Differential Movement in. (mm)	Effective Plasticity Index*	Reference	Foundation System	Description	Superstructure System**
<1/2 (<13)	<15	5, 7, 16, 66	Shallow	Continuous wall, individual spread footings	No limit
		25, 85, 53, 92-99	Reinforced and stiffened waffle mat	Residences and lightly loaded structures; on-grade 4-in. (100-mm) reinforced concrete slab with stiffening beams; 0.5% reinforcing steel; 8-12-in.-(200-300-mm-) thick beams; external beams thickened and extra steel stirrups added to tolerate high edge forces, as needed; dimensions adjusted to resist loading Beam Depth, in. (mm) / Beam Spacing, ft (m)	Semirigid; flexible; split construction
1/2-1 (13-25)	15-25	--	Light	16-20 (400-500) / 20-15 (6.0-4.5)	
1-2 (25-50)	26-40	--	Medium	20-24 (500-600) / 15-12 (4.5-3.6)	
2-4 (51-100)	>41	--	Heavy	25-30 (600-750) / 12-10 (3.6-3.0)	
No limit	--	5, 98	Thick, reinforced mat	Large, heavy structures; thickness of more than 1 ft (0.3 m)	No limit
No limit	--	7, 16, 17, 27, 57, 66, 68, 99	Beam on pier	Underreamed, reinforced, cast-in-place concrete piers; grade beams span between piers about 12 in. (300 mm) above ground level; suspended floors or on-grade first floor isolated from grade beams and walls	No limit

* See References 92, 96, 97 for definition; the weighted average PI in the top 15 ft (4.6 m) of soil below the stiffening beams.

** See Table 5 for description of superstructure systems.

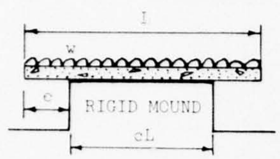
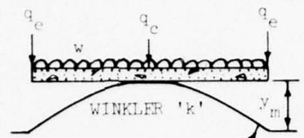
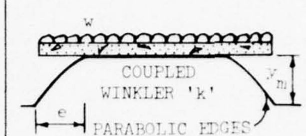
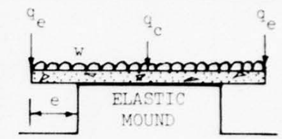
Table 5

Superstructure Systems

Superstructure System	Tolerable Deflection/Length Ratios	Reference	Description
Rigid	<1/1000	5, 24, 75, 101, 102	Precast concrete block, unreinforced brick, masonry or plaster walls, slab-on-grade
Semirigid	1/500 to 1/1000	6, 16, 37, 68, 75, 99, 103, 104	Reinforced masonry or brick reinforced with horizontal and vertical tie bars or bands made of steel bars or reinforced concrete beams; vertical reinforcement located on sides of doors and windows; slab-on-grade isolated from walls
Flexible	>1/500	5, 7, 16, 24, 27, 32	Steel, wood framing; brick veneer with articulated joints; metal, vinyl, or wood panels; gypsum board on metal or wood studs; vertically oriented construction joints; strip windows or metal panels separating rigid wall sections with 25-ft (7.5-m) spacing or less to allow differential movement; all water pipes and drains into structure with flexible joints; suspended floor or slab-on-grade isolated from walls (heaving and cracking of slab-on-grade probable and accounted for in design)
Split construction		16, 27, 32	Walls or rectangular sections heave as a unit (modular construction); joints at 25-ft (7.5-m) spacing or less between units and in walls; suspended floor or slab-on-grade isolated from walls (probable cracking of slab-on-grade); all water pipes and drains equipped with flexible joints; construction joints in reinforced and stiffened mat slabs at 150-ft (45-m) spacing or less and cold joints at 65-ft (20-m) spacing or less

Table 6

Summary of Relevant Design Methods[107]

DESIGN METHOD	BRAB (1968)[97]	LYTTON (1972)[94]	WALSH (1978)[95]	FRASER AND WARDLE (1975)[108]
ASSUMED SLAB ACTION	SIMPLIFIED THREE DIMENSIONAL	SIMPLIFIED THREE DIMENSIONAL	SIMPLIFIED THREE DIMENSIONAL	PRECISE THREE DIMENSIONAL
SLAB LOADING AND INITIAL MOUND SHAPE		 MOUND EQN. $y = \beta_m x^m$		 E, μ
DETERMINATION OF SLAB SUPPORT AREA COEFFICIENT "c"	FOR USA c HAS BEEN EMPIRICALLY RELATED TO CLAY TYPE AND WEATHER (ELSEWHERE $c = \frac{L - 2e}{L}$)	$c = \frac{m+1}{m+2}\left(\frac{m+1}{m} \cdot \frac{w}{k\, y_m}\right)^{\frac{1}{m+1}}$ $\frac{2e}{L} = 1 - \left(\frac{0.05}{y_m}\right)^{\frac{1}{m}}$	MATHEMATICALLY RELATED TO e, y_m, k AND w	$c = \frac{L - 2e}{L}$
CALCULATION OF "I"	FULLY CRACKED SECTION	UNCRACKED SECTION	PARTIALLY CRACKED SECTION	PARTIALLY CRACKED SECTION
CONCRETE LONG TERM "E" ($E_c = 28_d$ COMPR. STRENGTH)	SPECIFIED AS $E = 0.5\ E_c$	SPECIFIED AS $E = 0.5\ E_c$	NOT SPECIFIED. ADOPTED VALUE OF $E = 0.75\ E_c$	NOT SPECIFIED. ADOPTED VALUE OF $E = 0.75\ E_c$

LEGEND

c = support index

e = edge distance, ft

E = long-term modulus of concrete, tsf

E_c = modulus of concrete based on 28-day compressive strength, tsf

I = moment of inertia, ft^4

k = subgrade modulus, $tons/ft^3$

L = length of slab, ft

m = mound exponent

q_c = center load, tons/ft

q_e = edge load, tons/ft

w = average foundation pressure, tsf

y_m = maximum differential heave across the mound before slab-soil interaction, in.

β_m = constant characterizing mound shape

μ = Poisson's ratio

Table 7

Applications of Drilled Pier Foundations[7,27,71,113]

Applications	Advantages	Disadvantages
Absence of a shallow, stable, founding stratum; support of structures with piers drilled through swelling soils into zones unaffected by moisture changes	Personnel, equipment, and materials for construction usually readily available; rapid construction due to mobile equipment; careful inspection of excavated hole usually possible; noise level of equipment less than some other construction methods; low headroom needed	Interpretation of load tests requires expert knowledge and experience
Support of moderate to high column loads; high column loads with piers drilled into hard bedrock; moderate column loads with underreamed piers bottomed on sand and gravel	Excavated soil can be examined to check the projected soil conditions and profile; excavation possible for a wide variety of soil conditions	Careful design and construction required to avoid defective foundations; careful inspection necessary during construction; inspection of concrete after placement difficult
Support of light structures on friction piers	Heave and settlement at ground surface will normally be small for properly designed piers	Inadequate knowledge of design methods and construction problems can lead to improper design
Rigid limitations to structure deformations; differential heave or settlement exceeds 2-3 in. (50-80 mm); large lateral variations in soil conditions	Disturbance of soil minimized by drilling, thus reducing consolidation due to remolding compared to other methods of placing deep foundations	Construction techniques sometimes very sensitive to subsurface conditions: (1) susceptible to "necking" in squeezing ground, (2) difficult to concrete, requiring tremie if hole filled with slurry or water, (3) cement may wash out if water under artesian pressure, (4) pulling casing can disrupt continuity of concrete in shaft or displace/distort reinforcing cage
Structural configurations and functional requirements or economics preclude a mat or other foundation	A single shaft can carry very large loads often eliminating need for a cap	
	Changes in geometry (diameter, penetration, underream) can be made during construction if required by subsurface conditions	

Table 8

Failures Associated With Drilled Piers

Defect	Remarks
	Failures from Construction Techniques[113,115]
Discontinuities in the shaft	Often caused by cuttings left in the borehole prior to concreting. Too rapid pulling of casing can cause voids in the concrete. Groundwater hydrostatic pressure greater than concrete pressure. Inadequate spacing in steel reinforcement, inadequate concrete slump and workability.
Reduced diameter from caving soil	Caving or squeezing occurs along the shaft in cohesionless silt, rock flour, sand or gravel, and soft soils, especially below the water table. Coarse sands and gravels cave extensively during drilling and tend to freeze casing in place. Soft soils tend to close open boreholes. Raising the auger in soft soils may "suck" the borehole to almost complete closure.
Distortion of reinforcement	Distortion of steel reinforcement cages can occur while the casing is pulled. Horizontal bands should be placed around reinforcing steel.
Mode of Failure	**Remarks**
	Failures Attributed to Swelling Soil[82,117]
Subsoil wetting below base of shaft	Moisture may migrate down the concrete of the shaft from the surface or perched water tables into deeper desiccated zones, causing the entire pier to rise. Piers may also heave from a rising deep water table. Rise is sometimes avoided by increasing the pier length or placing the base in nonswelling soil or within a water table.
Uplift	Wetting of surrounding desiccated swelling clays can cause the shaft to rise and even fracture from excessive tensile stress. Rise can be reduced by placing an underreamed base in nonswelling soil, increasing steel reinforcement along the entire shaft length and underreamed base to resist the tensile stress, and providing sleeving to reduce adhesion between the shaft and soil.
Grade beams on swelling soil	Lack of an air gap between the surface of swelling soil and the grade beam can cause the grade beam to rise.
Lateral swell	Pier foundations have low resistance to damage from lateral swell. Downhill creep of expansive clays contribute to damaged pier foundations.

APPENDIX A: DETERMINATION OF SOIL SUCTION BY THERMOCOUPLE PSYCHROMETERS

Theory

1. The thermocouple psychrometer measures relative humidity in soil by a technique called Peltier cooling. By causing a current to flow through a single thermocouple junction in the proper direction, that particular junction will cool, causing water to condense on it when the dew-point temperature is reached. Condensation of this water inhibits further cooling of the junction. The voltage developed between the thermocouple and reference junctions is measured by the proper readout equipment.

2. The output of the thermocouple psychrometer (in microvolts) is calibrated by tests with salt solutions, such as potassium chloride (KCl) that produce a given relative humidity for known concentrations, as shown in the following tabulation:

Calibration Solutions

Gram-Formula Weight per 1000 g Water, M	Grams of KCl per 1000 ml water	Relative Humidity percent	Suction at 25°C, tsf
0.05	3.728	99.83	2.4
0.20	14.91	99.36	9.3
0.50	37.27	98.42	22.8
1.00	74.55	96.84	45.9

The relative humidities are converted to total suction by[48*]

$$\tau^{o} = -\frac{RT}{v_w} \ln \frac{p}{p_o} \qquad \text{(A1)}$$

* Superscript numerals in this and subsequent appendixes (and the mentioning of reference numbers) refer to similarly numbered sources listed in the References section at the end of the main text.

where

τ° = total suction free of external pressure except atmospheric pressure, tsf

R = universal gas constant, 86.81 cc-tsf/mole-Kelvin

T = absolute temperature, Kelvins

v_w = volume of a mole of liquid water, 18.02 cc/mole

p/p_o = relative humidity

p = pressure of water vapor, tsf

p_o = pressure of saturated water vapor, tsf

3. The total soil suction is defined as the sum of matrix τ_m° and osmotic τ_s suctions (Table A1). The matrix suction τ_m° is related to the geometrical configuration of the soil and structure, capillary tension in the pore water, and water sorption forces of the clay particles. The osmotic suction τ_s is caused by the concentration of soluble salts in the pore water. The matrix suction is pressure-dependent, whereas the osmotic suction is pressure-independent. The effect of the osmotic suction on swell is not well known, but an osmotic effect will be observed if the concentration of soluble salts in the pore water differs from that of the externally available water; i.e., swell may occur in the specimen if the external water contains less soluble salts than the pore water. The effect of the osmotic suction on swell behavior is assumed small compared with the effect of the matrix suction.

Procedure

4. Laboratory measurements to evaluate total suction may be made with the apparatus illustrated in Figure A1. Thermocouple psychrometers are inserted into 1-pt-capacity metal containers with the soil specimens and the assembly sealed with No. 13-1/2 rubber stoppers. The assembly is inserted into a 1- by 1- by 1.25-ft (0.3- by 0.3- by 0.4-m) chest capable of holding six 1-pt containers and insulated with 1.5 in.

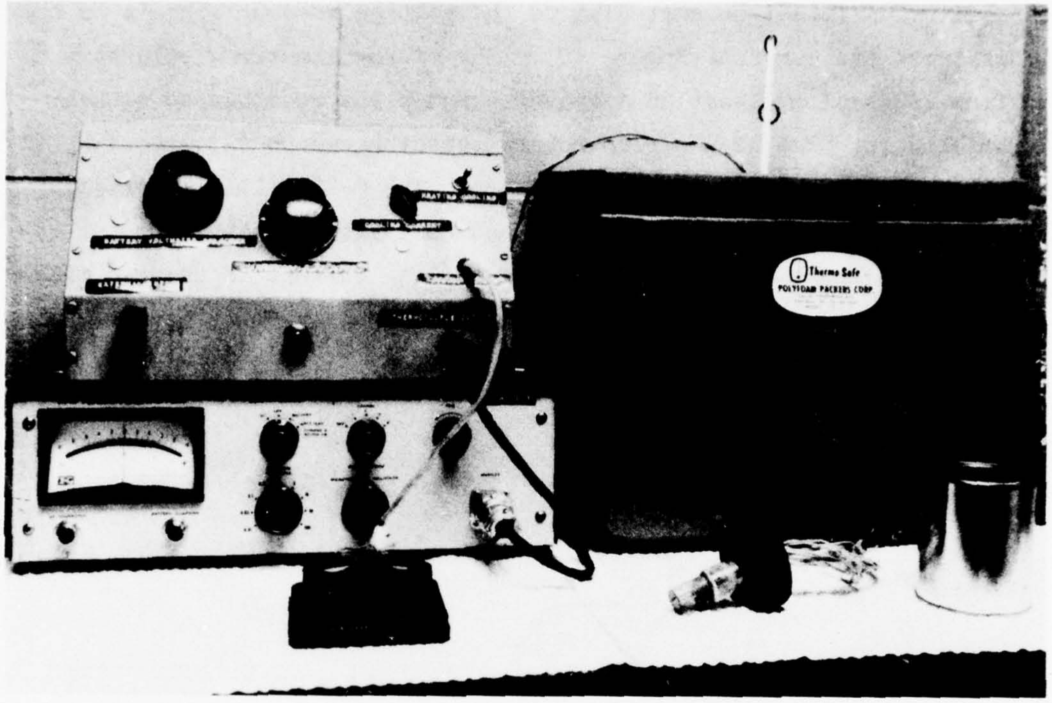


Figure A1. Monitoring system

(38 mm) of foamed polystyrene. Cables from the psychrometers are passed through a 0.5-in.- (13-mm-) diam hole centered in the chest cover. Temperature equilibrium is attained within a few hours after placing the lid. Equilibrium of the relative humidity in the air measured by the psychrometer and the relative humidity in the soil specimen is usually obtained within 24-48 hr.

5. The calibration curves of 12 commercial (Wescor) psychrometers acquired for the subject study were within 5 percent and could be expressed by

$$\tau^{\circ} = 2.65E_{25} - 1.6 \qquad \text{(A2)}$$

where

τ° = total suction, tsf

E_{25} = microvolts at 25°C

The monitoring system (Figure A1) includes a cooling circuit with the

capability of immediate switching to the voltage readout circuit on termination of the current (Figure A2). The microvoltmeter should have a maximum range of at least 30 microvolts and allow readings to within 0.1 microvolt. The 12-position rotary selector switch (2) allows up to 12 simultaneous psychrometer connections. The 0-25 milliammeter (3), two 1.5-volt dry cell batteries (4), and the variable potentiometer (5) form the cooling circuit. The optimum cooling current is about 8 milliammeters applied for 15 sec. The measurable range of suction varies from about 1-60 tsf (100-5700 kPa).

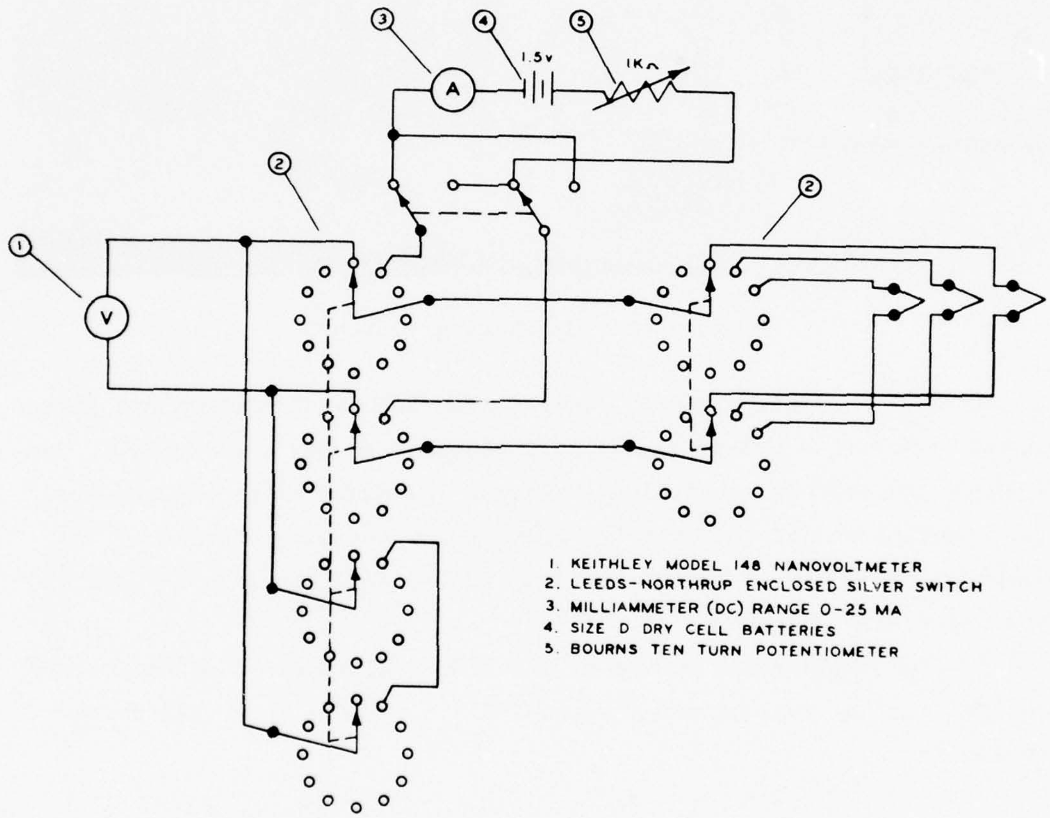


Figure A2. Electrical circuit for the thermocouple psychrometer

6. The readings can be taken at room temperature, preferably from 20 to 25°C, and corrected to E_{25} by

$$E_{25} = \frac{E_t}{0.325 + 0.027t} \tag{A3}$$

where E_t is the microvolt output at $t°C$. Placement of the apparatus in a constant temperature room will increase accuracy of the readings. Further details of this test procedure are available in References 19 and 43.

Characterization of Swell Behavior

7. The total soil suction-water content relationship of a particular soil is evaluated from multiple 1-in. (2-cm) pieces of the undisturbed sample. The pore water may be evaporated at room temperature for various periods of time up to about 48 hr from six undisturbed specimens; various amounts of distilled water may also be added to six other undisturbed specimens of each sample to obtain a 12-point water content distribution. Each specimen may be inserted into a 1-pt metal container with a thermocouple psychrometer for evaluation of the total soil suction by the above procedure. The dry density and void ratio of each undisturbed specimen may be evaluated by the water displacement method.[42]

Matrix suction

8. The 12-point total soil suction and water content relationship may be plotted as shown in Figure A3 for each undisturbed sample. An osmotic suction is indicated by a horizontally inclined slope at high water contents, and the magnitude may be estimated by noting the total soil suction at the high water contents. The matrix suction-water content relationship can be determined by subtracting the osmotic suction from the total soil suctions and expressing the result

$$\log \tau_m^o = A - Bw \tag{A4}$$

where

τ_m^o = matrix suction without surcharge pressure, tsf

A = ordinate intercept soil suction parameter, tsf

B = slope soil suction parameter

w = water content, percent dry weight

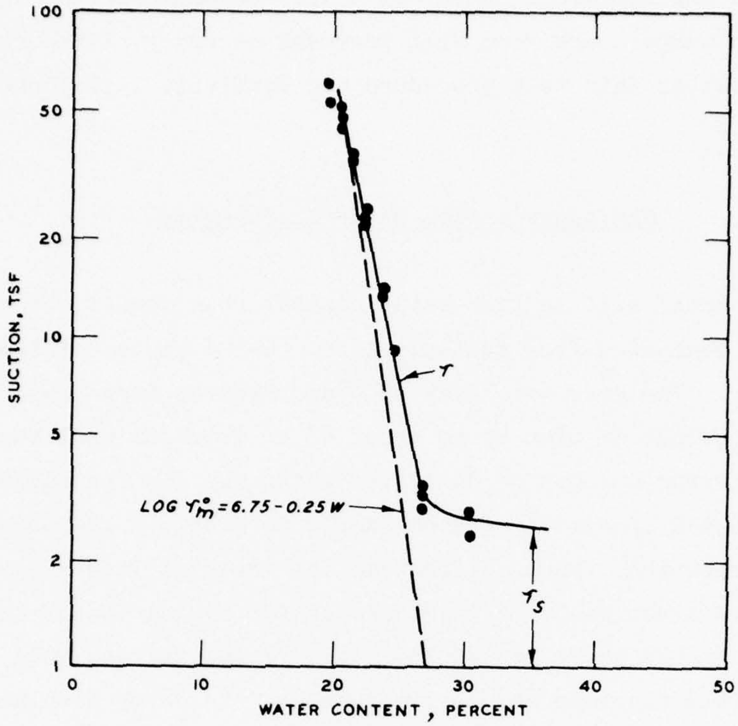


Figure A3. Suction-water content relationship of Lackland soil at 3.2-4.2 ft

Suction index

9. The suction index is analogous to the swell index of consolidometer swell tests, except that the suction index is evaluated with respect to the change in matrix soil suction rather than the change in pressure:[43]

$$\Delta e = C_\tau \log \frac{\tau_{mo}^o}{\tau_{mf}^o} \tag{A5}$$

where

Δe = change in void ratio

C_τ = $\alpha G_s/100B$, suction index

α = compressibility factor

G_s = specific gravity

τ_{mo}^o = initial matrix suction without surcharge pressure, tsf

τ_{mf}^o = final matrix suction without surcharge pressure, tsf

Suction indices are generally larger than swell indices and less than compression indices determined from consolidation tests.[43]

10. The initial matrix suction without surcharge pressure τ^{o}_{mo} may be evaluated using the soil suction test procedure and undisturbed specimens or may be calculated from Equation A4 and the initial water content. The final matrix suction without surcharge pressure τ^{o}_{mf} can be calculated assuming

$$\tau^{o}_{mf} = \bar{p}_f = p_f - u_w \tag{A6}$$

where

$\bar{p}_f$ = final mean normal effective pressure, tsf

p_f = final mean normal total pressure, tsf

The pore-water pressure u_w is found from Equation 1 or 2 in the main text. The consolidometer swell methods simply assume that $\bar{p}_f$ is equivalent with the final vertical effective pressure $\bar{\sigma}_v$.

Compressibility factor

11. The compressibility factor α is the ratio of the change in volume for a corresponding change in water content, i.e., the slope of the curve γ_w/γ_d plotted as a function of the water content where γ_w is the unit weight of water and γ_d is the dry density. Highly plastic soils commonly have α close to 1.0, while sandy and low plasticity soils commonly have α much less than 1.0. High compressibility factors can indicate highly swelling soils; however, soils with all voids filled with water also have an α equal to unity.

12. Figure A4 illustrates the compressibility factor calculated from laboratory data of a silty clay taken from a field test section near Clinton, Mississippi. Extrapolating the line to zero water content, as shown in the figure, provides an estimate of 1/R with

$$R = \frac{W_s}{V_o} \tag{A7}$$

where

R = shrinkage ratio

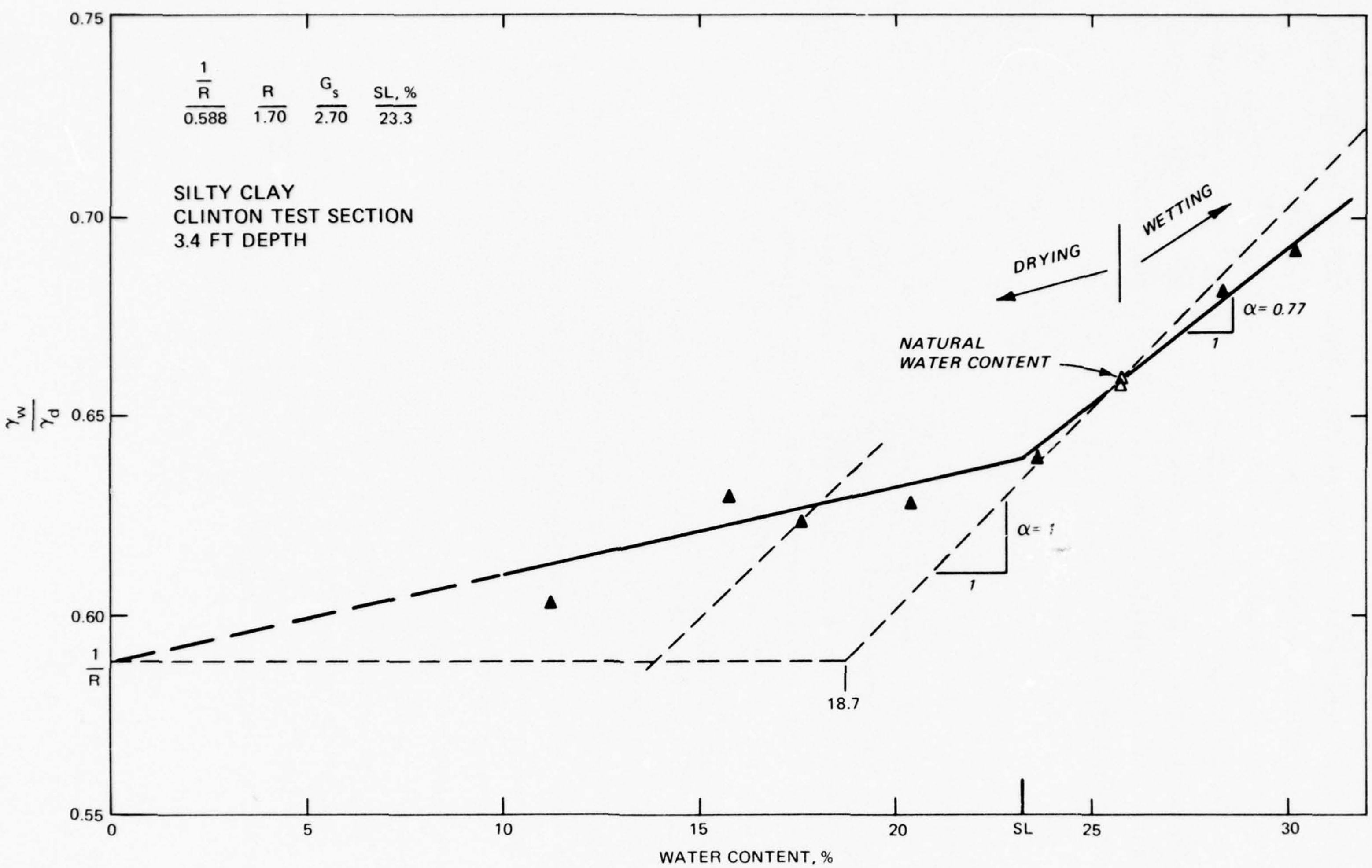


Figure A4. Illustration of the compressibility factor

W_s = mass of a specimen of oven-dried soil, g

V_o = volume of a specimen of oven-dried soil, cc

13. The shrinkage limit SL of the clay shown in Figure A4 may be taken at the abrupt change in slope of the curve, which is 23.3 percent. Calculation of the shrinkage limit by the equation given in EM 1110-2-1906,[42] i.e.,

$$SL = w - \left(\frac{V - V_o}{W_s}\right) 100 \tag{A8}$$

where

V = volume of the wet soil specimen, cc

may result in a SL that varies depending on the initial water content of the specimen. For example, if the initial water content is at the natural water content of 25.7 percent, then Equation A8 will give

$$SL = 25.7 - (0.658 - 0.588)\ 100 = 18.7$$

as shown in Figure A4. Other shrinkage limits may be evaluated by drawing straight lines with slope $\alpha = 1$ through other water content points. The actual shrinkage relationship of the soil does not indicate a SL at 18.7 percent. This shows the advantage of using the plot in Figure A4 and the compressibility factor to evaluate the volume-water content relationship for drying and wetting.

Suction swell pressure

14. The suction swell pressure is defined as the soil matrix suction without surcharge pressure that is in equilibrium with the soil when all voids are filled with water and the proportion of voids is given by the initial void ratio e_o. The suction swell pressure p_s may be evaluated from[43]

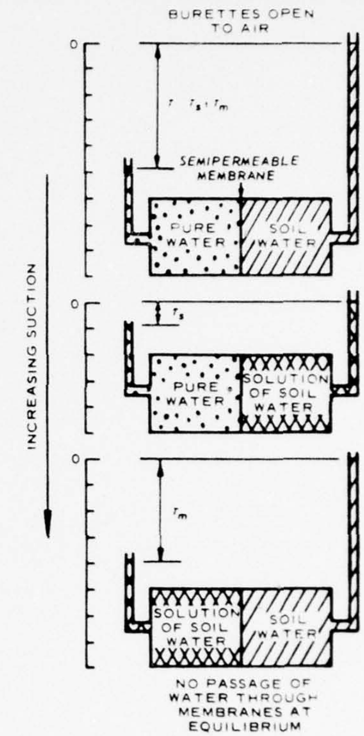
$$\log p_s = A - \frac{100Be_o}{G_s} \tag{A9}$$

The suction swell pressure is analogous to the swell pressure evaluated from results of consolidometer swell tests.

15. Equation A9, which calculates a swell pressure based on energy principles, is considered applicable where surface chemistry effects of clay particles are dominant. Inert particles in the soil, particularly gravels and pebbles, may preclude reliable calculations of swell pressure from Equation A9.

Table A1

Definitions of Suction

Term	Symbol	Definition*	Illustration
Total suction	τ	The negative gage pressure, relative to the external gas pressure** on the soil water, to which a pool of pure water must be subjected in order to be in equilibrium through a semipermeable (permeable to water molecules only) membrane with the soil water	
Osmotic (solute) suction	τ_s	The negative gage pressure to which a pool of pure water must be subjected in order to be in equilibrium through a semipermeable membrane with a pool containing a solution identical in composition with the soil water	
Matrix (soil water) suction	τ_m	The negative gage pressure, relative to the external gas pressure** on the soil water, to which a solution identical in composition with the soil water must be subjected in order to be in equilibrium through a porous permeable wall with the soil water	

* From Reference 48.

** The magnitude of the matrix suction is reduced by the magnitude of the external gas pressure. The osmotic suction is determined by the concentration of soluble salts in the pore water and can be given by $\tau_s = RT/v_w \log_e p/p_o$ where R is the universal gas constant, T is absolute temperature, v_w is volume of a mole of liquid water, p is vapor pressure of the pore-water extract, and p_o is vapor pressure of free pure water.

APPENDIX B: REMEDIAL MEASURES

1. Most damages from effects of swelling soils tend to be cosmetic, rather than structural. The results of an early statistical analysis[16] of damaged residences indicated that repairs are more economical than rebuilding as long as the structure remains structurally sound. Maintenance costs and frequency of repairs were observed to be greatest about 3 to 4 yr following the original construction. Overall maintenance expenses were minimized by repairing damages before extensive repairs were required, such as breaking out and replacing sections of walls. The choice of remedial measures should depend on the results of site and soil investigations. Investigation and repair are specialized procedures that usually require much expertise and experience.

2. All existing information on the foundation soil and design of the foundation and superstructure should be studied before proceeding with new soil investigations. Initial soil moisture at time of construction, types of soil, soil swell potentials, depth to the groundwater table, type of foundation and superstructure, and drainage system should be determined. Details of the foundation, such as loading pressures, size and length of footings, slab and pier reinforcing, are helpful. Drilling logs made during construction of pier foundations may help determine soil and groundwater conditions and details of pier foundations. Actual construction should be checked with plans of the design to determine compliance by the contractor.[71]

3. Types and locations of damage and when movements first became noticeable should be determined. Most cracks caused by differential heave are wider at the top than at the bottom. Nearly all lateral separation results from differential heave.[71] Diagonal cracks can indicate footing, drilled pier foundation movement, or lateral thrust from the doming pattern of heaving concrete slabs. Level surveys can be helpful to determine the trend of movement when prior survey records and reliable benchmarks are available. Excavations may be necessary to study damages to deep foundations, such as cracks in pier shafts from uplift forces.

4. The source of soil moisture that led to the differential heave should be determined to evaluate the cause of damages. Location of deep-rooted vegetation such as shrubs and trees, location and frequency of watering, inadequate slopes and ponding, seepage into foundation soil from surface or perched water, and defects in drain, water, and sewer lines can make important changes in soil moisture and can lead to differential heave.

5. Remedial measures can be more easily determined after the causes of differential heave have been pinpointed. Table B1 illustrates common remedial measures that can be taken. The structure should be allowed to adjust, following completion of remedial measures for up to a year before cosmetic work is done. The structure is seldom rebuilt to its original condition and in some instances, remedial measures have not been successful.[71]

6. Some remedial measures, such as mudjacking or construction of a series of spread footings or piers to repair and straighten damaged slabs-on-ground, may be several times the cost of the original foundation. Adequate soil investigations, landscaping, drainage, and foundation design are essential to avoid future prohibitive remedial repairs.

Table B1

Remedial Measures*

Measure	Description
Drainage	Slope ground surface (positive drainage) from structure; add drains for downspouts, outdoor faucets in areas of poor drainage and discharge away from foundation soil; provide subdrains if perched water tables or free flow of subsurface water are problems; provide flexible, water-tight utility connections.
Moisture stabilization	Remove and recompact (with impervious, nonswelling) backfill; install vertical and/or horizontal membranes around the perimeter; locate deep-rooted vegetation outside of moisture barriers; avoid automatic sprinkling systems in areas protected with moisture barriers; mix 4-8 percent lime in soil to reduce potential for swell or pressure-inject lime slurry.
Superstructure adjustments	Free slabs from foundation by cutting along foundation walls; provide slip joints in interior walls and door frames; reinforce masonry and concrete block walls with horizontal and vertical tie bars or reinforced concrete beams; provide fanlights over doors extended to ceiling.
Spread footings and deep foundation adjustments	Decrease footing size; underpin with piers; mudjack; reconstruct void beneath grade beams; eliminate mushroom at top of piers; adjust elevation by cutting the top or adding shims; increase footing or pier spacing to concentrate loading and to reduce angular distortion from differential heave between adjacent footings and piers.
Continuous wall foundation adjustments	Provide voids beneath portions of wall foundation; post-tension; reinforce with horizontal and vertical tie bars or reinforced concrete beams.
Reinforced and stiffened slab-on-ground adjustments	Mudjack; underpin with spread footings or piers to jack up the edge of slabs.

* From References 16, 21, 27, 37, 71, 99, 103, 105, and 106.

APPENDIX C: PREDICTION OF PIER MOVEMENT

Theory

1. The mechanism of pier movement, Table C1, is based on the premise that the uplift forces and resulting movements of the pier are caused by swelling pressures from soil wetting. The maximum swell pressures that can develop are functions of the void ratio or dry density of the surrounding soils.[43,71] The mechanism is consistent with the ideas of Chen,[71] except that the influence of final effective pressures of the soil and added restraining force from the bell are included. The analysis assumes that the interaction of stresses between skin friction and end bearing components is negligible. End bearing does not exist after pier uplift occurs. Predictions of pier movements from uplift forces are made for three cases: (a) moisture migrating down from the ground surface such as from rainfall, (b) moisture migrating from an intermediate zone such as from a relatively thin pervious sandy stratum, and (c) moisture migrating upward from below the pier such as from a rising water table.[125] Case 3 may also be used for the special case where X_a exceeds length L, but moisture migrates downward.

2. The formula for the restraining force P_r, Table C1, was developed after McAnally[124] who assumed a net upward bearing pressure from the bell of 7 times the shear strength τ_s. The shear strength is estimated by[7,66,69,120]

$$\tau_s = c' + K\bar{\sigma}_v \tan \phi \qquad \text{(C1)}$$

where

c' = effective cohesion, tsf

K = ratio of horizontal to vertical effective pressure

$\bar{\sigma}_v$ = effective vertical pressure, tsf

ϕ = effective angle of internal friction, degrees

3. The uplift force P_u is computed by

$$P_u = (p_s - \bar{p}_f)A_{act} \ ; \quad (p_s - \bar{p}_f) < f_s \qquad (C2)$$

$$P_u = f_s A_{act} \ ; \quad (p_s - \bar{p}_f) > f_s \qquad (C3)$$

where

p_s = swell pressure, tsf

$\bar{p}_f$ = final effective pressure, tsf

A_{act} = area over which the swell pressure is exerted on the pier shaft, ft^2

f_s = skin friction (Equation 9 in main text), tsf

If $(p_s - \bar{p}_f)$ is less than zero, then the uplift force does not exist, and it is replaced by a downdrag force exerted on the pier shaft and subsoils beneath the footing as discussed below.

4. The tension force T developed within the pier concrete from the uplift forces is compensated for the restraining effect of the final effective pressure $\bar{p}_f$ by

$$T = P - P_u \qquad (C4)$$

where P is the loading force exerted by the weight of the foundation and superstructure and P_u is given by Equation C2 or C3.

5. The force P_b exerted vertically downward at the bottom of the footing on the soil beneath the footing due to the loading force P is estimated by

$$P_b = P - (p_s - \bar{p}_f)L\pi D_p \ , \quad \left|p_s - \bar{p}_f\right| < f_s \qquad (C5)$$

$$P_b = P - f_s L\pi D_p \ , \quad \left|p_s - \bar{p}_f\right| > f_s \qquad (C6)$$

where L and D_p are the length and diameter of the pier shaft, respectively. The force P_b is set equal to zero, if Equation C5 or C6 results in negative values. If the swelling pressure is less than the vertical effective pressure, a dragdown force (negative skin friction) exerted by the surrounding soils is imposed on the shaft and the subsoil

beneath the footing adding to the loading force P .

Computer Program

Organization

6. The program HPIER for computing forces and pier movements from swelling soils is based on the above theory, and it is consistent with the format previously developed for the program ULTRAT for predicting total and rate of heave of structures constructed on expansive clay soils.[19] Computation of swell pressures and heaves are based on the mechanical swell and soil suction models described by Johnson.[19,43]

7. The program consists of a main routine and four subroutines. The main routine computes the effective vertical overburden and swell pressures, restraining force, tension force, and foundation pressure exerted on the subsurface soils beneath the footing. The subroutine MECH computes heave based on consolidometer swell tests. The subroutines SUCT and HSUCT compute heave based on the soil suction model. The subroutine PSAD sets up the proper depths in the soil profile for calculation of swell pressures and heaves. The program is set with statement PARAMETER NL=10, NQ=81 where NL is the maximum number of soils NMAT and NQ is the maximum number of nodal points NNP. The capacity of the program may be increased by increasing NL and NQ .

Input data

8. The program was prepared for time-sharing on the Honeywell series G600 computer. The input data are as follows:

Step	Data
1	The program will print: =. A description of the problem is recommended.
2	The program will print after carriage return: NOPT,NPROB,NSUCT,NNP,NBX,NMAT,DX =. Input the above variables, Table C2.
3	The program will print after carriage return: M,G,WC,EO,C,PHI =. Input the above variables, Table C2.
4A	If NSUCT=0, the program will print after carriage return:

Step	Data
	M,ALL,SP,CS,CC =. Input the above variables, Table C2, for soil M=1.
4B	If NSUCT=1, the program will print after carriage return: M,A,B,ALPHA,AKO,PI =. Input the above variables, Table C2, for soil M=1. The program will repeat steps 3 and 4 until all soils from M=1 to M=NMAT have been read into the computer.
5	The program will print after carriage return: ELEMENT,NO. OF SOIL =. Input 1,1 =. Input element, 2 for elements in increasing order for each increase in soil type M. =. Input NEL,NMAT as the last and deepest element for soil type M=NMAT.
6	The program will print after carriage return up to NPROB: PLOAD,XA,XF,AF,DP,DB,DGWT,IOPTION,KOPT =. Input the above variables, Table C2. Step 6 will be repeated following printing of the solution of a problem until the number of problems = NPROB.

Output data

9. If NOPT=1, all computed data will be printed:

Line	Data			
1	FORCE RESTRAINING UPLIFT=		EXCESS=	TONS
2	FORCE AT BOTTOM OF PIER=		TENSION=	TONS
3	HEAVE IN FEET: PIER=		SUBSOIL=	
4	ELEMENT DEPTH,FT	FRACTION HEAVE	EXCESS PORE PRESSURE,TSF	

If NOPT=0, line 4 and subsequent data tabulated for each soil element will not be printed. The nomenclature for the output data is defined in Table C3.

Application

Parametric analysis

10. The program HPIER was used to perform a limited parametric study of the movement and performance of piers 1.5 ft (457 mm) and 2.5 ft (762 mm) in diameter. The results of an analysis with the assumptions described in Table C4 led to the following empirical equations for estimating the maximum permissible depth of the active zone X_a :

Shaft Diameter D_p, ft	X_a, ft		
1.5	$0.71\left(\frac{D_b}{D_p}\right)^{2.5} + 0.5L + \frac{0.106P}{c_a}$	if $\phi = 0$	(C7)
2.5	$0.88\left(\frac{D_b}{D_p}\right)^{3.0} + 0.5L + \frac{0.064P}{c_a}$	if $\phi = 0$	(C8)
1.5	$0.88\left(\frac{D_b}{D_p}\right)^{2.5} + 0.7L + \frac{1.88P}{KL \tan \phi'}$	if $c_a = 0$	(C9)
2.5	$0.81\left(\frac{D_b}{D_p}\right)^{3.3} + 0.7L + \frac{1.14P}{KL \tan \phi'}$	if $c_a = 0$	(C10)

where

D_b = diameter of base, ft
D_p = diameter of shaft, ft
L = length, ft
P = loading force, tons
c_a = soil adhesion, tsf
K = ratio of horizontal to vertical effective pressure
ϕ' = effective angle of internal friction, degrees

11. If the actual swelling pressures are less than 1 tsf (96 kPa), then the maximum safe active zone will be deeper than those calculated by the Equations C7 to C10. These equations also show that the permitted X_a will be less if undrained strengths ($\phi = 0$) are valid than if soils with zero adhesion ($c_a = 0$) are assumed. For example, if piers are unloaded ($P = 0$), $D_b/D_p = 2.5$, and where the active zone equals or exceeds the lengths of the piers, lengths would have to exceed only 15 and 25 ft (4.5 and 7.6 m) for soils with $\phi = 0$, while lengths would be 31 and 52 ft (95 and 15.8 m) for soils with $c_a = 0$ for 1.5- and 2.5-ft- (457- and 762-mm-) diam shafts, respectively, to cause upward displacement from skin friction uplift forces. If no underream is used and

negligible loadings are comtemplated such as with residences and lightly loaded buildings, the pier length should be twice the depth of the active zone for soils with $\phi = 0$ or 1.5 times X_a for soils with $c_a = 0$.

12. Tension forces computed from the parametric analysis provided the basis for estimating the required percent steel A_S by

$$A_S = 0.094 \frac{Lc_a}{D_p} + 0.00275 \frac{L^2 K \tan \phi}{D_p} - 0.03 \frac{P}{D_p^2} \qquad \text{(C11)}$$

where the units in Equation C11 are the same as those in Equations C7-C10. The allowable stress in the steel reinforcing was assumed to be 60,000 psi (414 MPa).

Field tests

13. The program HPIER was used to analyze the performance of test piers 1 and 2 constructed at a test pier site on Lackland Air Force Base, Tex.[39] The input parameters, Table C5, were taken from results of constant volume (mechanical) swell and soil suction tests.[19,43] The strength parameter[7,39] c of 1 tsf was assumed equal to the soil adhesion c_a , and the coefficient K was taken as 1.0. The calculations indicate that total tension loads for the intact material shown in Figures C1 and C2 agree reasonably well with field data and are also reasonably consistent with results calculated by the Fort Worth District.[7]

14. The depth of the active zone X_a that would lead to pier heave was calculated by HPIER to be 27 ft for the intact material. Since the actual tension loads are significant along the pier shafts for lengths greater than 27 ft, HPIER predicts that the pier should be lifted upward from lateral skin friction uplift forces with the amount predicted varying from 54-89 percent of the heave of the adjacent soil, Table C6. Actual pier heave in excess of the soil heave observed at 34 ft of depth is about 69-76 percent of the adjacent soil heave.

15. An estimate of the reinforcing steel needed to resist the tension forces for zero loading force P is found from Equation C11:

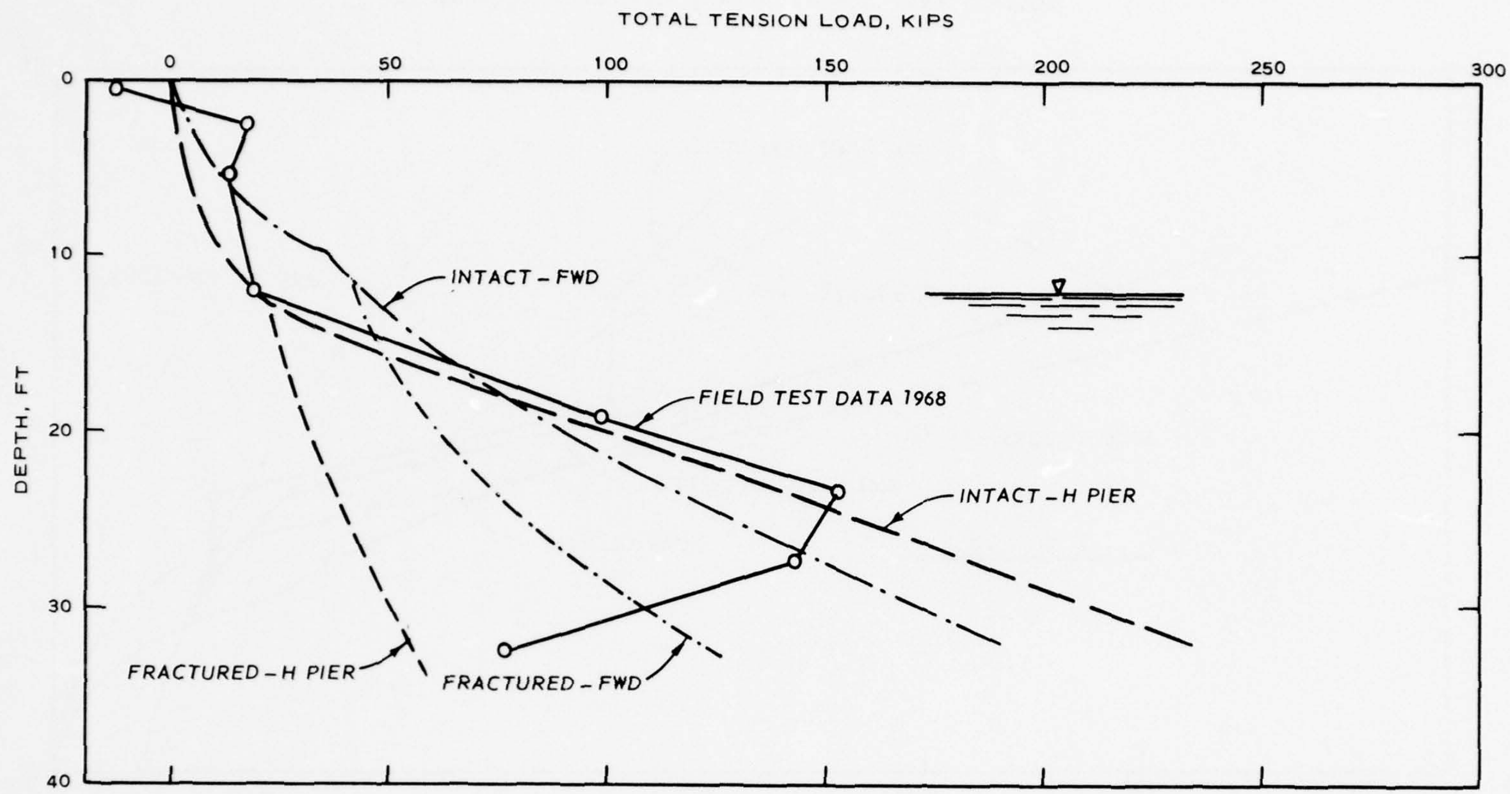


Figure C1. Tension loads of test pier 1, TP1

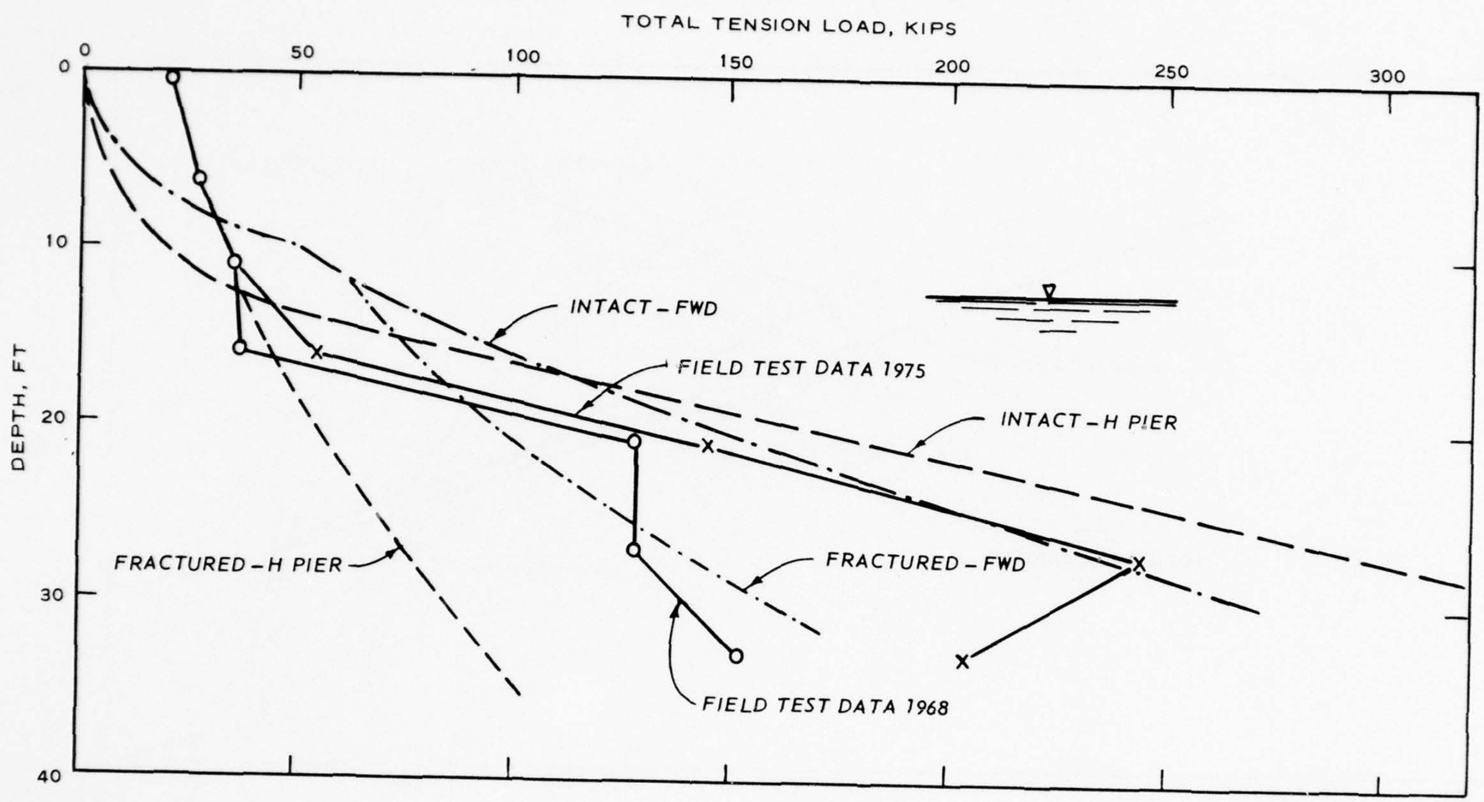


Figure C2. Tension loads of test pier 2, TP2

$$\text{TP1:}\quad A_s = \frac{0.094(30 - 12) \times 1}{1.5} + \frac{0.00274(30)^2 \times 1 \times 0.176}{1.5} = 1.4\% \qquad \text{(C12)}$$

$$\text{TP2:}\quad A_s = \frac{0.094(30 - 12) \times 1}{2.5} + \frac{0.00274(30)^2 \times 1 \times 0.176}{2.5} = 0.9\% \qquad \text{(C13)}$$

The actual amounts of steel placed in test piers 1 and 2 are 2 and 1 percent, respectively. These amounts should be satisfactory.

16. Table C7 presents a listing of the computer program. Table C8 presents an example of a program application for a suction model, and Table C9 presents an example of a program application for a CVS model.

Table C1

Prediction of Pier Movement

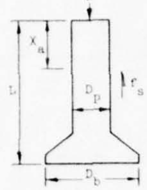
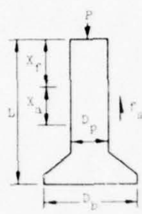
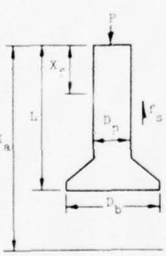
Case	Mechanism of Uplift	Sketch	Equations of Movement
1	The pier is lifted when the uplift force P_u given by the swell pressure $p_s - \bar{p}_f$ times the area over which the swell pressure is active A_{act} exceeds the restraining force P_r. The pier stops lifting when $P_u < P_r$ or the skin friction f_s times A_{act} is less than P_r		$\Delta_{pier} \leq 0$ if $(p_s - \bar{p}_f)A_{act} \leq P_r$, $P_r = P + f_s(L - X_a)\pi D_p + 7\tau_s \frac{\pi}{4}(D_b^2 - D_p^2)$ $\Delta_{pier} \leq 0$ if $f_s A_{act} \leq P_r$ $\Delta_{pier} = X_a C_s \log \frac{p_s}{P_r/A_{act}}$ if $p_s > \frac{P_r}{A_{act}} > \bar{p}_f$ $\Delta_{pier} = X_a C_s \log \frac{p_s}{\bar{p}_f}$ if $p_s > \bar{p}_f > \frac{P_r}{A_{act}}$ $\Delta_{soil} = X_a C_s \log \frac{p_s}{\bar{p}_f}$, $A_{act} = X_a \pi D_p$
2	Same mechanism as case 1 except that soil from the ground surface to depth X_f does not swell and contributes no uplift. Case 2 converges to case 1 when $X_f = 0$		$\Delta_{pier} \leq 0$ if $(p_s - \bar{p}_f)A_{act} \leq P_r$, $P_r = P + f_s(L - X_a)\pi D_p + 7\tau_s \frac{\pi}{4}(D_b^2 - D_p^2)$ $\Delta_{pier} \leq 0$ if $f_s A_{act} \leq P_r$ $\Delta_{pier} = X_a C_s \log \frac{p_s}{P_r/A_{act}}$ if $p_s > \frac{P_r}{A_{act}} > \bar{p}_f$ $\Delta_{pier} = X_a C_s \log \frac{p_s}{\bar{p}_f}$ if $p_s > \bar{p}_f > \frac{P_r}{A_{act}}$ $\Delta_{soil} = X_a C_s \log \frac{p_s}{\bar{p}_f}$, $A_{act} = X_a \pi D_p$
3	The pier is lifted a distance equal to the vertical swelling of the soil beneath the pier as wetting ascends to the base of the pier. The pier is lifted further as soil wetting ascends above the base when the uplift force P_u exceeds the restraining force P_r. The pier stops lifting when $P_u < P_r$ or $f_s A_{act} < P_r$		$\Delta_{pier} \leq 0$ if $(p_s - \bar{p}_f)A_{act} \leq P_r$, $P_r = P + 7\tau_s \frac{\pi}{4}(D_b^2 - D_p^2)$ $\Delta_{pier} \leq 0$ if $f_s A_{act} \leq P_r$ $\Delta_{pier} = (X_a - L)C_s \log \frac{p_s}{\bar{p}_f} + (L - X_f)C_s \log \frac{p_s}{P_r/A_{act}}$ if $p_s > \frac{P_r}{A_{act}} > \bar{p}_f$ $\Delta_{pier} = (X_a - X_f)C_s \log \frac{p_s}{\bar{p}_f}$ if $p_s > \bar{p}_f > \frac{P_r}{A_{act}}$ $\Delta_{soil} = (X_a - X_f)C_s \log \frac{p_s}{\bar{p}_f}$, $A_{act} = (L - X_f)\pi D_p$

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Table C2

Nomenclature of Input Data

Symbol	Step	Description
		Problem Parameters
NOPT	2	Option for amount of output: =0 for forces and total heave; =1 for forces, total heave, and the fraction and excess pore pressure at each depth interval
NPROB	2	Number of cases with the same material properties, pier length, and soil profile
NSUCT	2	Option for model: =0 for mechanical swell model; =1 for soil suction model
NNP	2	Total number of nodal points, NEL+1
NBX	2	Number of nodal point at the bottom of the pier
NMAT	2	Total number of different soil layers
DX	2	Increment of depth, ft
		Physical Properties
M	3	Number of soil layer
G	3	Specific gravity of soil layer M, G_s
WC	3	Initial water content of soil layer M, w_o percent
EO	3	Initial void ratio of soil layer M, e_o
C	3	Soil cohesion c or undrained shear strength, tsf
PHI	3	Ratio of horizontal to vertical effective pressure times the tangent of the effective angle of internal friction, Ktan
		Swell Characterization by the Mechanical Swell Model
M	4A	Number of soil layer
ALL	4A	Liquid limit of soil layer M, LL percent
SP	4A	Swell pressure of soil layer M, p_s tsf
CS	4A	Swell index of soil layer M, C_s
CC	4A	Compression index of soil layer M, C_c
		Swell Characterization by the Soil Suction Model
M	4B	Number of soil layer
A	4B	Intercept of suction-water content relationship of soil layer M, tsf
B	4B	Slope of suction-water content relationship of soil layer M
ALPHA	4B	Compressibility factor of soil layer M, α
AKO	4B	Ratio of total horizontal to vertical pressure of soil layer M, K_T
PI	4B	Plasticity index of soil layer M, PI percent
		Element Characterization
ELEMENT	5	Number of soil element
NO. of SOIL	5	Number of soil layer M
NEL	5	Total number of soil elements
NMAT	5	Total number of soil layers
		Problem Characterization
PLOAD	6	Loading force on pier, P tons
XA	6	Depth of the active zone, X_a ft
XF	6	Depth from ground surface to the depth that the active zone begins, X_f ft
AF	6	Reduction factor of skin friction term (Equation 8), α_f
DP	6	Diameter of pier shaft, D_p ft
DB	6	Diameter of base of pier, D_b ft
DGWT	6	Depth to the groundwater table, ft
IOPTION	6	Equilibrium moisture profile: =0 for saturation; =1 for hydrostatic
KOPT	6	Source of moisture: =1 from ground surface; =2 from an intermediate layer; =3 from below base of pier. Heave of soil adjacent to the pier is computed if a zero is added after each of these integers: i.e., 10, 20, or 30 for KOPT cases 1, 2, or 3, respectively

Table C3

Nomenclature of Output Data

Symbol	Line	Description
FORCE RESTRAINING UPLIFT	1	Force restraining uplift, P_r tons
EXCESS	1	Restraining force P_r - uplift force P_u, tons
FORCE AT BOTTOM OF PIER	2	Force exerted on soil beneath the pier footing, P_b tons
TENSION	2	Maximum tension in pier, T tons
HEAVE IN FEET:		
PIER	3	Uplift of pier, ft in KOPT = 1, 2, or 3. Uplift of soil adjacent to pier, ft if KOPT = 10, 20, or 30. Does not include heave beneath base of pier
SUBSOIL	3	Uplift of soil beneath base of pier, ft
ELEMENT	4	Number of element
DEPTH,FT	4	Depth of center of element, ft
FRACTION HEAVE	4	$(e_f - e_o)/(1 + e_o)$ for each element
EXCESS PORE PRESSURE,TSF	4	Mechanical swell model: $(p_s - \overline{p}_f)$ for each element soil suction model: $(\tau_{mo} - \tau_m)$ for each element

Table C4

Assumptions of Parametric Analysis

1. Source of moisture was from the ground surface (Case 1, Table C1).
2. Equilibrium moisture profile was saturated (pore-water pressure = 0 in the active zone).
3. Swell pressures exceed 1 tsf.
4. Depth of the groundwater table was below base of the pier.
5. Soil adhesion c_a values were less than 1 tsf with friction angle $\phi = 0$, and ϕ values were less than 20 deg with $c_a = 0$.
6. Ratio of horizontal to vertical pressures were equal to one.
7. The moist unit weight was 122.5 lb/ft^3.

Table C5

Input Parameters for Field Test Piers

	TP1	TP2
Physical Dimensions		
D_p , ft	1.5	2.5
D_b , ft	3.0	4.0
L , ft	34.0	35.0

Depth, ft	G_s	w_o , %	e_o	c_a , tsf	c_a , (degrees)	p_s , tsf	C_s	C_c	LL, %
Constant Volume (Mechanical) Model									
0.0-8.0	2.68	17.9	0.800	0	20	2.20	0.045	0.27	70
8.0-13.0	2.71	23.8	0.745	0	30	0.70	0.030	0.27	49
13.0-30.0	2.75	31.0	0.838	1(0)*	10	2.40	0.052	0.20	75
30.0-	2.76	29.0	0.884	1(0)*	10	2.85	0.048	0.13	80
						A	B	Alpha, α	PI, %
Soil Suction Model									
0.0-8.0	2.75	32.0	0.880	0	20	4.544	0.135	1.00	40
8.0-13.0	2.75	30.0	0.825	0	30	5.044	0.167	0.26	14
13.0-30.0	2.76	30.0	0.828	1(0)*	10	5.859	0.179	1.00	55
30.0-	2.76	30.0	0.828	1(0)*	10	6.135	0.185	1.00	55

* Residual or fractured material.

Table C6

Upward Movement of Test Piers

Depth of Adjacent Soil ft	Difference in Level Observations from 1966, ft			Percent of Adjacent Soil Heave		
	1968	1971	1975	Observed 1975	Predicted CVS	Predicted Suction
Test Pier 1	0.006	0.076	0.128	69	68-89	54-81
1.0	0.004	0.025	0.111			
14.0	0.000	0.088	0.181			
34.0	0.000	0.031	0.058			
Test Pier 2	0.008	0.079	0.170	76	68-89	54-81
1.0	0.010	0.079	0.213			
14.0	0.008	0.109	0.250			
34.0	0.004	0.058	0.103			

Table C7

Listing of Computer Program

```
6744T 01  02-15-79   09.749
C   PREDICTION OF PIER MOVEMENT
C   BASED ON CONSTANT VOLUME SWELL/SWELL OVERBURDEN/SUCTION
C   DEVELOPED BY L. D. JOHNSON
    PARAMETER NL=10,NO=81
    COMMON A(NL),B(NL),G(NL),WC(NL),EO(NL),SP(NL),ALL(NL),
    PI(NL),C(NL),RHI(NL),CS(NL),CC(NL),ALPHA(NL),[illegible](NL),P(NO),
    IE(NO,1),N1,N2,NBX,NEL,IOPTION,KOPT,MOPT,BORT,GAW,DX,DXX,
       DGWT,PRE,DP,RII,XA,XF,Q,TFI,DELH
  READ 3
  3    FORMAT(30H
    GAW=0.03125
    PI=3.14159265
    NP=1
    PRINT 5
    5     FORMAT(32HNOPT,NPROB,NSUCT,NNP,NBX,NMAT,DX)
          READ,NOPT,NPROB,NSUCT,NNP,NBX,NMAT,DX
    NEL=NNP-1
    14 PRINT 10
    10 FORMAT(15HM,G,WC,EO,C,PHI)
       READ,M,G(M),WC(M),EO(M),C(M),PHI(M)
    IF(NSUCT.EQ.1)GO TO 25
    PRINT 12
    12   FORMAT(14HM,ALL,SP,CS,CC)
         READ,M,ALL(M),SP(M),CS(M),CC(M)
    GO TO 20
    25    PRINT 8
     8 FORMAT(18HM,A,B,ALPHA,AKO,PI)
         READ,M,A(M),B(M),ALPHA(M),AKC(M),PI(M)
    IF(ALPHA(M).LE.0.)GO TO 16
    GO TO 20
    16    ALPHA(M)=.0275*PI(M)-.125
    IF(RI(M).LE.5.)ALPHA(M)=0.0
        IF(PI(M).GE.40.)ALPHA(M)=1.
    20    IF(NMAT-M)26,27,14
    26    PRINT 17,M
    17    FORMAT(20H  ERROR IN MATERIAL ,I5)
    STOP
    27 L=0
    PRINT 30
    30    FORMAT(19HELEMENT,NO. OF SOIL)
    40    READ,N,IE(N,1)
    50    L=L+1
    IF(N-L)60,60,70
    70    IE(L,1)=IE(L-1,1)
    GO TO 50
    60    IF(NEL-L)80,80,40
    80    CONTINUE
    85   PRINT 90
    90   FORMAT(/,38HPLOAD,XA,XF,AF,DP,DB,DGWT,IOPTION,KOPT)
        READ,PLOAD,XA,XF,AF,DP,DB,DGWT,IOPTION,KOPT
        IF(NSUCT.EQ.0.AND.IOPTION.GT.1)IOPTION=1
C   CALCULATION OF EFFECTIVE OVERBURDEN PRESSURE
```

(Continued)

Table C7 (Continued)

```
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      P(1)=0.0
      DXX=DX
      DO 100 I=2,NNP
      MTYP=IE(I-1,1)
      WCC=WC(MTYP)/100.
      GAMM=G(MTYP)*GAW*(1.+WCC)/(1.+EO(MTYP))
      IF(DXX.GT.DGWT)GAMM=GAMM-GAW
      P(I)=P(I-1)+DX*GAMM
      DXX=DXX+DX
  100 CONTINUE
      IF(KOPT.GT.5)GO TO 220
C CALCULATION OF RESTRAINING FORCE
      CON=DX*PI*DP*AF
      P1=0.0
      PR1=0.0
      PS1=0.0
      IF(KOPT.EQ.3)GO TO 122
      AN1=XA/DX
      N1=IFIX(AN1)+1
      N2=NBX-1
      IF(N1.GT.N2)GO TO 122
      DO 120 I=N1,N2
      MTYP=IE(I,1)
      IF(NSUCT.EQ.0)GO TO 115
      TAUI=A(MTYP)-B(MTYP)*WC(MTYP)
      SP(MTYP)=10.**TAUI
      SPRE=A(MTYP)-100.*B(MTYP)*EO(MTYP)/G(MTYP)
      SPRE=10.**SPRE
      IF(SPRE.LT.SP(MTYP))SF(MTYP)=SPRE
  115 PS1=PS1+SP(MTYP)*CON
      PR=(P(I)+P(I+1))/2.
      PR1=PR1+PR*CON
      P1=P1+(PR*PHI(MTYP)+C(MTYP))*CON
  120 CONTINUE
  122 MAT=IE(NBX-1,1)
      PH=PHI(MAT)
      CU=(P(NBX)*SIN(PH)/COS(PH))+C(MAT)
  125 PRE=PLOAD+P1+7.*CU*PI*(DB**2.-DP**2.)/4.
C CALCULATION OF EXCESS RESTRAINING FORCE AT BOTTOM OF PIER
      P2=0.0
      PR2=0.0
      PS2=0.0
      MOPT=0
      CALL PSAD
      DO 150 I=N1,N2
      MTYP=IE(I,1)
      IF(NSUCT.EQ.0)GO TO 145
      TAUI=A(MTYP)-B(MTYP)*WC(MTYP)
      SP(MTYP)=10.**TAUI
      SPRE=A(MTYP)-100.*B(MTYP)*EO(MTYP)/G(MTYP)
      SPRE=10.**SPRE
      IF(SPRE.LT.SP(MTYP))SP(MTYP)=SPRE
```

(Continued)

(Sheet 2 of 7)

Table C7 (Continued)

```
6744T 01  02-15-79   09.749
2040    145  PS2=PS2+SR(MTYP)*CON
2050           PR=(P(I)+P(I+1))/2.
2060           PR2=PR2+PR*CON
2070        P2=P2+(PR*PHI(MTYP)+C(MTYP))*CON
2080     150 CONTINUE
2090           PST=PS1+PS2
2100           PRT=PR1+PR2
2110         CAT=P1+P2
2120          DPSR=PS2-PR2
2130           Q=PRE-DPSR
2140           IF(DPSR.GT.P2)Q=PRE-P2
2150          PRINT 160,PRE,Q
2160     160  FORMAT(/,25HFORCE RESTRAINING UPLIFT=,E10.5,9H  EXCESS=,
2170&         F10.5,6H  TONS)
2180C    CALCULATION OF FOUNDATION PRESSURE BENEATH FOOTING
2190        PSTPRT=PST-PRT
2200           QQ=PLOAD-PSTPRT
2210           IF(PSTPRT.GT.CAT)QQ=PLOAD-CAT
2220           T=PLOAD-DPSR
2230           IF(DPSR.GT.P2)T=PLOAD-P2
2240           PRINT 170,QQ,T
2250     170   FORMAT(25HFORCE AT BOTTOM OF PIER= ,F10.5,9H TENSION= ,
2260&          F10.5,6H  TONS)
2270        IF(QQ.LT.0.0)GO TO 220
2280        QQ=4.*QQ/(PI*DB**2.)
2290        BPRES=QQ-P(NBX)
2300         DXX=0.0
2310         DO 200 I=NBX,NNP
2320         IF(I.EQ.NBX)GO TO 201
2330        TP=1.+(DP/(2.*DXX))**2.
2340         TP=TP**1.5
2350        P(I)=P(I)+BPRES*(1.-1./TP)
2360         GO TO 205
2370    201 P(I)=P(I)+BPRES
2380     205 DXX=DXX+DX
2390     200 CONTINUE
2400C    ADJUST FINAL PRESSURES TO EFFECTIVE FOR MECH MODEL
2410      220  DXX=DX
2420         DO 250 I=2,NNP
2430         AI=I-1
2440         BN=DGWT/DX-AI
2450           IF(NSUCT.GT.0)GO TO 275
2460           IF(DXX.LE.DGWT.AND.ICPTION.EQ.5)P(I)=P(I)+BN*DX*GAW
2470           GO TO 280
2480      275  IF(DXX.GT.DGWT)P(I)=P(I)-BN*DX*GAW
2490     280 DXX=DXX+DX
2500     250 CONTINUE
2510     290 IF(NOPT.EQ.0)GO TO 300
2520          IF(KOPT.GT.5)GO TO 303
2530          IF(Q.GT.0.0.AND.NBX.EQ.NNP)GO TO 300
2540     303    PRINT 305
2550     305 FORMAT(/,33HELEMENT  DEPTH,FT  FRACTION HEAVE,
```


(Continued)

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```
     26H  EXCESS PORE PRESSURE,TSF)
 380  IF(NSUCT.EQ.0)CALL MECH
      IF(NSUCT.GT.0)CALL SUCT
      NP=NP+1
      IF(NP.GT.NPROB)GO TO 310
      GO TO 85
 310 STOP
     END
C
C
      SUBROUTINE MECH
     PARAMETER NL=10,NQ=81
      COMMON A(NL),B(NL),G(NL),WC(NL),EO(NL),SP(NL),ALL(NL),
     RI(NL),C(NL),PHI(NL),CS(NL),CC(NL),ALPHA(NL),AKO(NL),P(NQ),
     IE(NQ,1),N1,N2,NBX,NEL,IOPTION,KOPT,MOPT,NOPT,GAW,DX,DXX,
     DGWT,PRE,DP,PII,XA,XF,Q,TFI,DELH
      DELH1=0.0
        IF(KOPT.GT.5)GO TO 45
      IF(Q.GT.0.0)GO TO 50
  45  .MOPT=1
      CALL PSAD
      DO 10 I=N1,N2
      MTYP=IE(I,1)
      PR=(P(I)+P(I+1))/2.
     CA=SP(MTYP)/PR
        IF(PR.LT.PRE.AND.KOPT.LT.5)CA=SP(MTYP)/PRE
        E=EO(MTYP)+CC(MTYP)*ALOG10(CA)
        IF(PRE.LT.SP(MTYP))E=EC(MTYP)+CS(MTYP)*ALOG10(CA)
        IF(PR.LT.SP(MTYP).AND.KOPT.GT.5)E=EB(MTYP)+CS(MTYP)*ALOG10(CA)
      DEL=(E-EO(MTYP))/(1.+EO(MTYP))
     IF(NOPT.EQ.0)GO TO 40
     DELP=SP(MTYP)-PR
     PRINT 200,I,DXX,DEL,DELP
  40  DELH1=DELH1+DX*DEL
      DXX=DXX+DX
  10  CONTINUE
        IF(DELH1.LT.0.0.AND.KOPT.LT.5)DELH1=0.0
  50  DELH2=0.0
      NNP=NEL+1
      IF(NBX.EQ.NNP)GO TO 175
     DXX=FLOAT(NBX)*DX-DX/2.
      DO 100 I=NBX,NEL
      MTYP=IE(I,1)
      PR=(P(I)+P(I+1))/2.
     CA=SP(MTYP)/PR
     E=EO(MTYP)+CC(MTYP)*ALOG10(CA)
     IF(PR.LE.SP(MTYP))E=EO(MTYP)+CS(MTYP)*ALOG10(CA)
      DEL=(E-EO(MTYP))/(1.+EO(MTYP))
      IF(NOPT.EQ.0)GO TO 125
      DELP=SP(MTYP)-PR
      PRINT 200,I,DXX,DEL,DELP
 125 DELH2=DELH2+DX*DEL
```

(Continued)

Table C7 (Continued)

```
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            DXX=DXX+DX
      100 CONTINUE
      175 PRINT 210,DELH1,DELH2
      200 FORMAT(I5,F10.2,F15.5,5X,F15.5)
      210 FORMAT(/,21HHEAVE IN FEET:  PIER=,F8.5,10H  SUBSOIL=,
            F8.5)
            RETURN
            END
C
C
            SUBROUTINE SUCT
            PARAMETER NL=10,NQ=81
            COMMON A(NL),B(NL),G(NL),WC(NL),EO(NL),SP(NL),ALL(NL),
          RI(NL),C(NL),PHI(NL),CS(NL),CC(NL),ALPHA(NL),AKO(NL),P(NQ),
            IE(NQ,1),N1,N2,NBX,NEL,IOPTION,KOPT,MOPT,KOPT,GAW,DX,DXX,
          DGWT,PRE,DP,PII,XA,XF,Q,TFI,DELH
            NN=NEL+1
            IF(IOPTION.EQ.2)GO TO 5
            GO TO 7
        5   MATNEL=IE(NEL,1)
            FNNP=(1.+2.*AKO(MATNEL))/3.
          SUCTI=A(MATNEL)-B(MATNEL)*WC(MATNEL)
            SUCTI=10.**SUCTI
          TFI=SUCTI-P(NN)*FNNP*ALPHA(MATNEL)
        7   DELH1=0.0
              IF(KOPT.GT.5)GO TO 45
          IF(Q.GT.0.0)GO TO 50
       45     MOPT=1
            CALL PSAD
            CALL HSUCT
            DELH1=DELH
       50   DELH2=0.0
            IF(NBX.EQ.NN)GO TO 175
          DXX=FLOAT(NBX)*DX-DX/2.
            N1=NBX
            N2=NEL
          MOPT=0
            CALL HSUCT
          DELH2=DELH
      175 PRINT 200,DELH1,DELH2
      200 FORMAT(/21HHEAVE IN FEET:  PIER=,F8.5,10H  SUBSOIL=,
            F8.5)
            RETURN
            END
C
C
            SUBROUTINE HSUCT
            PARAMETER NL=10,NQ=81
            COMMON A(NL),B(NL),G(NL),WC(NL),EO(NL),SP(NL),ALL(NL),
          RI(NL),C(NL),PHI(NL),CS(NL),CC(NL),ALPHA(NL),AKO(NL),P(NQ),
            IE(NQ,1),N1,N2,NBX,NEL,IOPTION,KOPT,MOPT,KOPT,GAW,DX,DXX,
          DGWT,PRE,DP,PII,XA,XF,Q,TFI,DELH
```

(Continued)

(Sheet 5 of 7)

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```
      ANEL=FLOAT(NEL)*DX
      DELH=0.0
      DO 10 I=N1,N2
  MTYP=IE(I,1)
  F=(1.+2.*AKO(MTYP))/3.
  AI=I-1
  BN=(DGWT/DX)-AI
  BO=BN-1.
  TF=0.0
  IF(IOPTION.EQ.1.OR.DXX.GT.DGWT)TF=(BN+BO)*DX*GAW/2.
  IF(IOPTION.EQ.2)TF=TFI+(ANEL-DXX)*GAW
  PR=(P(I)+P(I+1))/2.
  ALP=ALPHA(MTYP)
  IF(DXX.GT.DGWT)ALP=1.0
  TAUF=TF+PR*F*ALP
        IF(KOPT.GT.5)GO TO 5
     IF(TAUF.LT.PRE.AND.MOPT.EQ.1)TAUF=PRE
   5    IF(TAUF.GT.0.000001)GO TO 15
  PRINT 20,TAUF,I
  20   FORMAT(31HNEGATIVE FINAL EFFECTIVE STRESS,
&  F10.5,12H  IN ELEMENT,I5)
  GO TO 35
  15 TAUI=A(MTYP)-B(MTYP)*WC(MTYP)
  TAUI=10.**TAUI
      IF(MOPT.EQ.0)GO TO 18
      SPRE=A(MTYP)-100.*B(MTYP)*EO(MTYP)/G(MTYP)
      SPRE=10.**SPRE
      IF(SPRE.LT.TAUI)TAUI=SPRE
  18  TI=TAUI-ALP*PR*F
     UINIT=TI-TF
     CT=ALPHA(MTYP)*G(MTYP)/(100.*B(MTYP))
  CT=CT/(1.+EO(MTYP))
  RTAU=TAUI/TAUF
     DEL=CT*ALOG10(RTAU)
     IF(DEL.LT.0.0.AND.DXX.GT.DGWT)DEL=DEL/ALPHA(MTYP)
     IF(DEL.LT.0.0.AND.TI.LT.0.0)DEL=DEL/ALPHA(MTYP)
        IF(NOPT.EQ.0)GO TO 33
     PRINT 30,I,DXX,DEL,UINIT
  30    FORMAT(I5,F10.2,F15.5,5X,F15.5)
  33 DELH=DELH+DX*DEL
  35 DXX=DXX+DX
  10    CONTINUE
        IF(DELH.LT.0.0.AND.MOPT.EQ.1.AND.KOPT.LT.5)DELH=0.0
      RETURN
      END
C
C
      SUBROUTINE PSAD
      PARAMETER NL=10,NQ=81
      COMMON A(NL),B(NL),G(NL),WC(NL),EO(NL),SP(NL),ALL(NL),
&     PI(NL),C(NL),PHI(NL),CS(NL),CC(NL),ALPHA(NL),AKO(NL),P(NQ),
&      IE(NQ,1),N1,N2,NBX,NEL,IOPTION,KOPT,MOPT,NOPT,GAW,DX,DXX,
```

(Continued)

```
6744T 01   02-15-79    09.749

        DGWT,PRE,DP,PII,XA,XF,Q,TFI,DELH
           IF(KOPT.EQ.1.OR.KOPT.EQ.10)GO TO 10
           IF(KOPT.EQ.2.OR.KOPT.EQ.20)GO TO 15
         AN1=XF/DX
         N1=IFIX(AN1)+1
         N2=NBX-1
          DXX=XF+DX/2.
           IF(MOPT.EQ.0.OR.KOPT.GT.5)GO TO 20
         ANBX=FLOAT(NBX)*DX-DX
         PRE=PRE/(ANBX*PII*DP)
         GO TO 20
   10   AN2=XA/DX
          DXX=DX/2.
         N1=1
         N2=AN2
        AN3=AN2*DX
        N3=NBX-1
        IF(N2.GT.N3)AN3=FLOAT(N3)*DX
        IF(N2.GT.N3)N2=N3
        IF(MOPT.EQ.1.AND.KOPT.LT.5)PRE=PRE/(AN3*PII*DP)
         GO TO 20
   15    AN1=XF/DX
         AN2=XA/DX
         N1=IFIX(AN1)+1
         N2=AN2
          DXX=XF+DX/2.
        AN3=AN2*DX
        N3=NBX-1
        IF(N2.GT.N3)AN3=FLOAT(N3)*DX
        IF(N2.GT.N3)N2=N3
           IF(MOPT.EQ.0.OR.KOPT.GT.5)GO TO 20
        PRE=PRE/((AN3-XF)*PII*DP)
   20    CONTINUE
        RETURN
        END
```

Table C8

Example of Program Application, Suction Model

```
RUN
=TEST PIER 1     INTACT MATERIAL     SUCTION MODEL     DGWT=12 FT
NOPT,NPROB,NSUCT,NNP,NBX,NMAT,DX
=0,20,1,69,69,4,.5
M,G,WC,EO,C,PHI
=1,2.75,32.,.88,0.,.364
M,A,B,ALPHA,AKO,PI
=1,4.544,.135,1.,1.,40.
M,G,WC,EO,C,PHI
=2,2.75,30.,.825,0.,.577
M,A,B,ALPHA,AKO,PI
=2,5.044,.167,.26,1.,14.
M,G,WC,EO,C,PHI
=3,2.76,30.,.828,1.,.176
M,A,B,ALPHA,AKO,PI
=3,5.859,.179,1.,2.,55.
M,G,WC,EO,C,PHI
=4,2.76,30.,.828,1.,.176
M,A,B,ALPHA,AKO,PI
=4,6.135,.185,1.,2.,55.
ELEMENT,NO. OF SOIL
=1,1
=17,2
=27,3
=61,4
=68,4

PLOAD,XA,XF,AF,DP,DB,DGWT,IOPTION,KOPT
=0.,1.,0.,1.,1.5,3.,12.,0,1

FORCE RESTRAINING UPLIFT= 175.79709  EXCESS= 175.74534  TONS
FORCE AT BOTTOM OF PIER= -129.56935 TENSION=  -0.05175  TONS

HEAVE IN FEET:  PIER= 0.       SUBSOIL= 0.
```

Table C9

Example of Program Application, CVS Model

```
RUN
=TEST PIER 1    INTACT MATERIAL    CVS MODEL    DGWT=12 FT
NOPT,NPROB,NSUCT,NNP,NBX,NMAT,DX
=0,20,0,69,69,4,.5
M,G,WC,EO,C,PHI
=1,2.68,17.9,.8,0.,.364
M,ALL,SP,CS,CC
=1,70.,2.2,.045,.27
M,G,WC,EO,C,PHI
=2,2.71,23.8,.745,0.,.577
M,ALL,SP,CS,CC
=2,49.,.7,.03,.27
M,G,WC,EO,C,PHI
=3,2.75,31.,.838,1.,.176
M,ALL,SP,CS,CC
=3,75.,2.4,.052,.2
M,G,WC,EO,C,PHI
=4,2.76,29.,.884,1.,.176
M,ALL,SP,CS,CC
=4,80.,2.85,.048,.13
ELEMENT, NO. OF SOIL
=1,1
=17,2
=27,3
=61,4
=68,4

PLOAD,XA,XF,AF,DP,DB,DGWT,IOPTION,KOPT
=0.,5.,0.,1.,1.5,3.,12.,0,1

FORCE RESTRAINING UPLIFT= 172.44676  EXCESS= 171.27056  TONS
FORCE AT BOTTOM OF PIER= -127.74035 TENSION=  -1.17619  TONS

HEAVE IN FEET:  PIER= 0.       SUBSOIL= 0.
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APPENDIX D: NOTATION

A	Ordinate intercept soil suction parameter, tsf
A_{act}	Area over which swell pressure is exerted, ft^2
A_p	Bearing area of pier base, ft^2
A_s	Bearing area of pier shaft, ft^2
A_S	Reinforcing steel, percent
B	Slope soil suction parameter
c	Strength intercept (cohesion) of the assumed straight-line Mohr envelope, tsf
c'	Effective cohesion, tsf
c_a	Soil adhesion, tsf
c_u	Undrained shear strength, tsf
c_{vs}	Average effective coefficient of swell, ft^2/day
C	Support index
C_c	Compression index
C_s	Swell index
C_τ	Suction index, $\alpha G_s/100B$
D_b	Diameter of pier base, ft
D_p	Diameter of pier shaft, ft
e	Edge lift-off distance, void ratio
e_o	Initial void ratio
e_f	Final void ratio
E	Long-term creep modulus of concrete, tsf
E_c	Modulus of concrete based on 28-day compression strength, tsf
E_s	Modulus of elasticity of soil, tsf
E_t	Microvolts at $t^\circ C$
E_{25}	Microvolts at $25^\circ C$
f_s	Ultimate skin friction or shaft resistance, tsf
F	Fraction of potential heave
F_H	Reduction factor to account for pressure at depth H
G_s	Specific gravity
I	Moment of inertia, ft^4
k	Subgrade modulus, tons/ft^3

k_s	Average effective coefficient of permeability of saturated soil, ft/day
K	Ratio of horizontal to vertical effective stress
L	Pier length, ft; length of slab, ft
LL	Liquid limit, percent
m	Mound exponent
N_c	Bearing capacity factor
N_q	Bearing capacity factor
p	Pressure of water vapor, tsf
p_f	Final mean normal total pressure, tsf
p_o	Pressure of saturated water vapor, tsf
p_s	Swell pressure, tsf
$\bar{p}_f$	Final effective pressure, tsf
P	Loading force, tons
P_b	Force exerted vertically downward on soil beneath the footing, tons
P_r	Restraining force
P_u	Uplift force, tons
PI	Plasticity index, percent
q_c	Center load, tons/ft
q_e	Edge load, tons/ft
q_p	Ultimate base resistance, tsf
q_s	Normal stress acting on pier shaft, tsf
q_u	Unconfined compression strength, psi
Q_o	Ultimate total load, tons
Q_p	Ultimate base load, tons
Q_s	Ultimate shaft load, tons
R	Universal gas constant, 86.81 cc-tsf/mole-Kelvin; shrinkage ratio
SL	Shrinkage limit, percent
S_p	Potential swell, percent
t	Time, days; degrees C
T	Tension force in pier, tsf; absolute temperature, degrees Kelvin
u_w	Pore-water pressure, tsf
u_{wa}	Pore-water pressure at depth of the active zone X_a, tsf

v_w	Volume of a mole of liquid water, 18.02 cc/mole
V	Volume of a wet soil specimen, cc
V_o	Volume of a oven-dried soil specimen, cc
w	Water content, percent dry weight; average foundation pressure, tsf
w_o	Initial water content, percent dry weight
W_s	Mass of a oven-dried specimen, g
X	Depth, ft
X_a	Depth of the active zone, ft
X_f	Depth of inactive soil at the ground surface, ft
y_m	Maximum differential swell, in.
α	Compressibility factor
α_f	Reduction coefficient in skin resistance depending on type of pier and soil conditions
β	Relative stiffness length, ft
β_m	Constant characterizing mound shape
γ_d	Dry density, tons/ft^3
γ_w	Unit weight of water, 0.03125 tons/ft^3
Δe	Change in void ratio
μ	Poisson's ratio
$\bar{\sigma}_v$	Effective vertical stress, tsf
τ_{mf}	Final in situ matrix suction, tsf
τ_{mo}	Initial in situ matrix suction, tsf
τ_{nat}	Natural soil suction
τ_s	Osmotic suction, tsf; shear strength, tsf
τ^o	Total soil suction free of external pressure except atmospheric, tsf
τ_m^o	Matrix soil suction free of external pressure except atmospheric, tsf
τ_{mf}^o	Final matrix suction without surcharge pressure, tsf
τ_{mo}^o	Initial matrix suction without surcharge pressure, tsf
ϕ	Angle of internal friction, degrees
ϕ'	Effective angle of internal friction, degrees
ψ	Angle of friction between soil and pier shaft, degrees

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Johnson, Lawrence D
Overview for design of foundations on expansive soils / by Lawrence D. Johnson. Vicksburg, Miss. : U. S. Waterways Experiment Station ; Springfield, Va. : available from National Technical Information Service, 1979.
60, [40] p. : ill. ; 27 cm. (Miscellaneous paper - U. S. Army Engineer Waterways Experiment Station ; GL-79-21)
Prepared for Office, Chief of Engineers, U. S. Army, Washington, D. C., under RDT&E Work Unit AT40 EO 004.
References: p. 50-60.

1. Clays. 2. Expansive clays. 3. Expansive soils. 4. Foundation design. 5. Soil swelling. 6. Structural design. I. United States. Army. Corps of Engineers. II. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Miscellaneous paper ; GL-79-21.
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